SANTA CLARA VALLEY WATER DISTRICT

HYDROLOGY

Documentation

for the

East Flood Control Zone

Prepared by

A. D. Saah, P. E.
Associate Civil Engineer
Project Development Branch

Under the Direction of
Managements of Water Supply Planning
and
ction Units

4

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INTRODUCTION

This report summarizes the work completed on the hydrology within the Santa Clara Valley Water District's East Flood Control Zone. Presented in the report are recommended design flows from Berryessa, Coyote, Silver and Upper Penitencia Creeks and each of their respective tributaries. They are divided into several hydrologic studies each containing a table of contents, general information about the physical and hydrometeorological features of the basin, and a discussion of the procedures followed. This last item includes the analysis of recorded data as well as recorded and reconstituted data. It also includes a comparative analysis of the regional and the site-specific statistics.

The choice of the final design flows is based on findings from the above results. It is the intent of this report, as well as other similar reports, to present a documentation of the hydrologic assumptions made, together with steps taken in the mathematical calculations. Each creek was originally treated separately. Inasmuch as the study has been combined in terms of flood control zones, some redundancy is to be expected.

The following is a list of the tributaries analyzed in these studies:

Coyote Creek Hydrology

Coyote Creek
Fisher Creek
Packwood Creek
Upper Silver Creek

Lower Silver Creek Hydrology

Cribari Creek
Evergreen Creek
Flint Creek
Fowler Creek
Lower Silver Creek
Miguelita Creek
North Babb Creek
Norwood Creek
Quimby Creek
Ruby Creek
South Babb Creek
Thompson Creek
Yerba Buena Creek

Upper Penitencia Creek Hydrology

Upper Penitencia Creek

Berryessa Creek Hydrology

Berryessa Creek
Calero Creek
East Penitencia Creek
Ford and Wrigley Ditches
Los Coches Creek
Lower Penitencia Creek
Piedmont Creek
Sierra Creek
Sweigert Creek
Tularcitos Creek

SUMMARY OF METHODS AND PROCEDURES

General

The District has established its flood protection criterion for watershed areas greater than four square miles based on flows which has a 1% chance of being equaled or exceeded in any one year. For watershed areas between one and four square miles the criterion is the 2% flood and for watershed areas between 360 acres and one square mile it is the 4% flood.

The accompanying reports represent the 1977-78 update of the design flood flows for use in the design of flood control facilities in Santa Clara County. They present documentation for the calculation of the 1% and 10% design flood flows. Other design values (2% and 4%) are not presented herein; they could be obtained upon request. These values were calculated using the District's methods and procedures as augmented and modified by the recommendations of the panel of expert hydrologists* convened to review the methods in 1976 (Ref. 8). It was assumed in the derivation of these design flows that the channel system is free from major constrictions or significant flood storage (reservoirs are handled separately) and that the floodwaters at the design level are completely contained within channel banks or relatively narrow floodway limits.

The District's method of determining the design peaks and volumes of floods utilized all available sources of information to derive the most accurate values possible. In general, this method began with the use of Regional Regression Equations to compute peaks and volumes. These peaks and volumes were compared with the results of an analysis of reliable recorded streamflow data when such exist. The values were also compared with the results of a rainfall-runoff model before the final "design flow" classification is given. Where design flows are presently affected by reservoirs or by urbanization, special analyses accounting for their effects were performed. Finally, the design flows were compared with the results of other studies and with previously determined design flows. The results of such comparison are useful to further define the design values.

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*The panel members were Leo Beard, Dave Dawdy, Neil Grigg, Douglas L. James and Vujica Yevjevich. Their report, "Review of Basic Hydrology Methodology for Flood Control" Santa Clara Valley Water District, 1976, is available from the District library.

Hydrographs were derived utilizing the Corps of Engineers' standard project storm and the Clark Synthetic Unit Hydrograph technique. The peaks and volumes of these hydrographs were then adjusted to agree with those values derived from regional or local statistics. The flood hydrographs obtained this way are considered representative for use in the design of most flood control facilities. However, where flood traps (small volume peak skimmers) are being considered, a sensitivity analysis of the design relative to hydrograph shape should be conducted.

Following is a more detailed disucssion of these methods. Full documentation of the analyis of each stream is available in separate reports.

Regional Regression Analysis

The backbone of the District's hydrology is the application of the Regional Regression Equations (1976) for the determination of flood peaks and volumes. These equations were obtained by analyzing recorded streamflow data (peak flows and their respective volumes) from stations within and outside the District. The data were correlated, expanded, and the resultant statistics adjusted to obtain the dependent variables (mean, standard deviation, and 1% flow). These statistics were then regressed to the physical characteristics of their respective basins (independent variables) by means of a mathematical process called Multiple Linear Regression. The regression equations chosen had minimal standard errors of estimates and maximum determination coefficients. These equations are applicable to all unurbanized locations within the District to generate statistics for the determination of design flood peaks and volumes.

Gaged and Ungaged Locations

At gaged locations the statistics from the regression equations were compared with the site-specific statistics obtained from recorded data when such data were usable. The statistics from recorded data were obtained using the methods presented in the Water Resources Council Bulletin #17 (Ref. 2). If there were significant differences between the regional and site-specific results, then new statistics (weighted with respect to their respective standard errors) were calculated. These statistics, together with a Pearson Type III distribution a -0.6 regional skew coefficient or a weighted one, were used to determine the frequency regime.

At ungaged and rural locations, the results of the regression equations are considered to provide the best available information and, hence, are to be used for analysis and design.

Techniques Used for Developing Flood Hydrographs

Pattern runoff hydrographs are obtained from rainfall using Clark's Synthetic Unit Hydrograph method (as developed by the Corps of Engineers' HEC-1 computer program). This rainfall-runoff transformation involves the determination of loss rates, synthetic unit hydrograph parameters, spatial distribution of rainfall over the watershed and temporal distributions of rainfall rates within one storm. The first two items are usually obtained from projections of optimized results of recorded rainfall and its respective runoff data. The averaging of rainfall over the basin is usually done by the regular Theissen weighing operations. The temporal distribution is determined generally by using the Corps of Engineers' Standard Project Storm for Northern California.

If the peaks and volumes of the pattern hydrographs obtained are drastically different from the regional or site-specific results, then the reasons were investigated and, if justified, further adjustments were made. In this study, the rainfall-runoff-generated peaks and volumes are found to generally agree with regional or statistical values when average rainfall and area configurations were present. To avoid the uncertainties inherent in the rainfall-runoff transformation, the peaks and volumes generated from this transformation are balanced by regional values.

The balanced hydrographs obtained above are then combined with other hydrographs and routed downstream. The routing was done using linear methods (e.g., Muskingum by computing k when $x = 0$) if routing through channel system, or nonlinear methods (modified Pulse) if significant storage is available.

Rainfall distribution, time of concentration and the various routing operations are the items that define the shape of pattern hydrographs, with rainfall distribution being the foremost. The effect of the shape of these hydrographs, given constant peaks and volumes, may become significant when flow trapping is being considered as a flood protection measure. An in-depth analysis of the shape was not attempted as a part of this work.

Regulated Watersheds

In regulated watersheds, regulated by reservoirs, the design flows are assumed as inflows to the reservoirs. The initial storage levels of the reservoirs that coincide with the design flows of certain frequencies are computed from the stage-duration frequency (S.D.F.) analysis of daily recorded reservoir volumes. The Corps of Engineers' Coincidental Frequency Analysis (discussed in Leo Beard's "Statistical Methods") procedures were used to solve this compounded probability problem. At most reservoirs, the S.D.F. curves after 1968 were used because, since that time, the reservoirs have retained more water in storage longer to enhance recreation.

Urban Hydrology

The above procedures are applicable mainly to rural watersheds. For urban watersheds, the methodology changes and the regional regression equations do not apply.

Urban hydrology deals with the effects due to two major changes in the watershed, namely:

1. Increased imperviousness
2. Channelization

The increase in imperviousness is caused by rooftops, streets, parking lots, driveways, and other man-made ground surface cover. Channelization includes the flow of water in gutters, storm drains, improved channels, etc. An increase in the imperviousness tends to reduce the loss rates and increase the runoff volumes and peaks, while channelization may either increase or decrease the peaks. It depends on the magnitude of the flood and the storage-discharge capacity of the various channels or storm drain systems that carry the flood.

To determine the effect of increased imperviousness, the urban area of every watershed was divided into subareas tributary to the major storm drains. These subareas are then transformed into equivalent rectangular subareas where the length is equal to the length of an equivalent street. The ratio of imperviousness (which is a measure of land use) is then used to subdivide this equivalent subarea into two sub-subareas of pervious and impervious parts.

In order to calculate the separate inflow hydrographs from the two sub-subareas, the unit hydrograph procedure was employed using separate times of concentration for both the pervious and impervious parts. The routing coefficient is calculated from the length and width of these areas. The loss rates, which are the third set of parameters needed for the computation of the one percent flood hydrograph, are assumed to be 30 percent for the pervious part and five percent for the impervious part. Since most of the floodwater from urban areas is conveyed to flood control channels by the storm drain system, these hydrographs are routed through the storm drain system utilizing the modified Pulse routing method. This routing requires storage-discharge curves for the system of streets, gutters, manholes, and storm drains. Two actual urban areas, one of low-density residential and another of high-density residential and commercial, were used to develop equivalent generalized storage facilities and storage-discharge curves. The resultant curves were transformed to reflect unit areas, and then compared with each other. Since the two curves are very similar, an average smooth curve was chosen and used.

Because most of the urban area in Santa Clara Valley is in the flatland and similar storm drain design criteria were used throughout, it is assumed that the unit storage-discharge curve that was obtained from the above study is applicable to all the urban areas in the Valley.

The above method is detailed in a paper presented before the National Symposium on Urban Hydrology in 1976.

Upper 80 Percent Confidence Limits

As a District policy, the upper 80 percent confidence value for the 1% design flow rate is used as one input to define freeboard criteria. It is defined as the value at which there is a 90 percent chance that the design flow is equal to less than the given value. It relates to the standard errors associated with the statistical inferences of information about populations from very limited samples. These values are calculated using the Non-Central Student "t" Distribution as shown in the Water Resources Council Bulletin #17.

Previous Studies and Coordinated Results

As a part of the design flood flow determination procedure, all available hydrologic analyses done on any watershed were reviewed and considered.

These previous analyses were sometimes the result of an in-house effort and other times the result of efforts of other agencies.

Because of the nature of random events, especially in the case of small samples, the results obtained from previous analysis were not necessarily congruent with the new values. Therefore, an in-depth search of the adequacy of the various results and methods involved was done.

As a result of this study, some of the adopted values were supported not only by District-developed methods and techniques but also by those of the Corps of Engineers and others.

Further Information

The attached information is only a summary report of that available. Additional information, available upon request from the Hydrologic Library, is listed below:

1. Hydrographs, either plotted or numerical, at all locations.

2. Runoff from intermediate areas such as urban runoff.

3. Recorded streamflow data at most gaged locations.

4. Stochastically estimated data at most gaged stations.

5. A full spectrum of statistical analysis of the above data which include the investigation of outliers, the incorporation of historic flows, and the adjustment of statistics and probabilities.

6. Statistical analysis of rainfall data on all gaged locations.

7. Complete descriptions and documentation of computations done for other streams in the County.

8. Other supporting documents of the methods like the 1976 Panel of Experts Report, the 1976 Regional Regression Study, Paper on Urban Hydrology and Reports by Zone, discussing the hydrology in detail.

Acknowledgments

In an effort to accelerate the District's hydrology study schedule to match the requirements of the Federal Insurance Administration, George S. Nolte and Associates performed the hydrology study of Berryessa and parts of Upper Penitencia Creeks under District direction.

E. S. Watson, Division Engineer, was very instrumental in directing this work and reviewing the first draft of this report.

COYOTE CREEK HYDROLOGY

Determination of Design Flows

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GENERAL INFORMATION

Coyote Creek drains the western part of the Diablo Mountains. The watershed (Figure 1-1) covers the area from the County boundary in the north to Kickham Park in the south. The upper reaches of the creek are controlled by two reservoirs: Coyote (23,600 acre-feet, built in 1936) and Anderson (91,280 acre-feet, built in 1950).

The watershed tributary to Coyote Reservoir (118 square miles) is drained by the east and middle forks of Coyote Creek that join near Soda Spring Canyon to form the Upper Coyote Creek. From there the creek flows south to a point near the Canada De Los Osos where it changes direction and flows north to Coyote Reservoir. The average slope of these reaches is 1%.

The watershed tributary to Anderson Reservoir, excluding the Coyote Reservoir Watershed, is drained by Shingle, Las Animas, and San Felipe Creeks. The flow pattern of these creeks is southerly until it reaches Anderson Reservoir. The average slope of these reaches is 1%.

Downstream of Anderson, the flow of Coyote Creek is mainly towards the north where is travels a distance of more than 50 miles at a 0.4% slope before reaching the bay. Fisher, Silver, and Penitencia Creeks join Coyote in the lower parts.

The upper watershed of Coyote Creek is mainly rural while the lower parts, downstream from the intersection of U. S. Highway 101, are mainly urban.

The existing capacity of the main channel of Coyote Creek ranges from 12,000 cfs downstream from Anderson Reservoir and 13,000 cfs at Ford Road (S. C. No. 58) to 2,000 cfs downstream of Route 237.

The Coyote Basin has warm, dry summers and mild, wet winters. The mean annual precipitation ranges with elevation from a low of 13 inches near the bay to a maximum of 28 inches near Mount Sizer (3,217 feet).

The ground cover is mainly pastureland with light forest cover in the valleys and canyons.

The objective of this report is to document the hydrology work done to determine design flows. It will discuss the use of regional regression equations and the analysis of recorded data together with the special treatment of regulated and urban watershed.

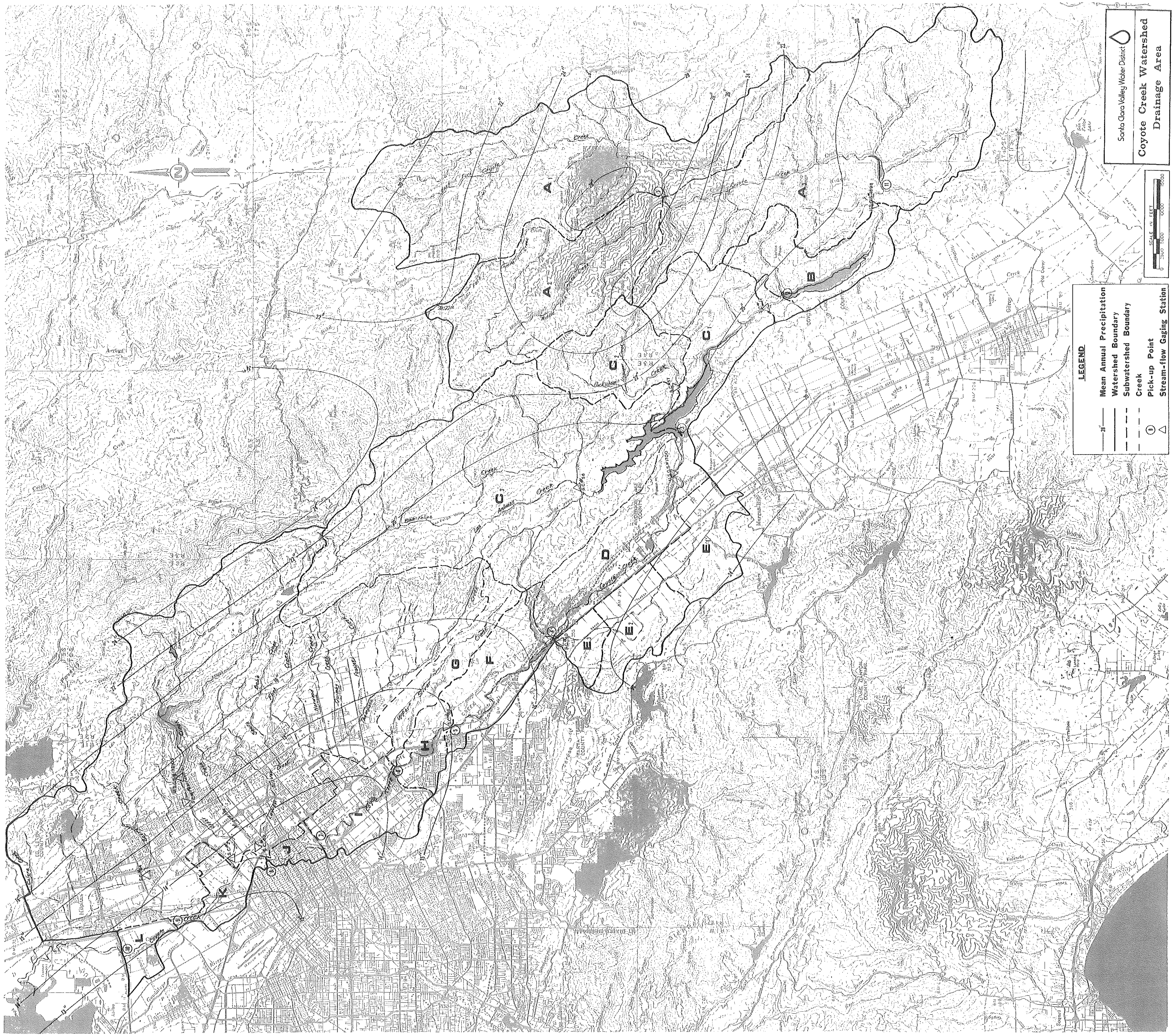
Santa Clara Valley Water District
Coyote Creek Watershed
Drainage Area
SCALE IN FEET
0
2500
5000
10000
15000
LEGEND
28″
Mean Annual Precipitation
Watershed Boundary
Subwatershed Boundary
Creek
Pick-up Point
Stream-flow Gaging Station
A
A
A
B
C
C
C
D
E
E
E
F
G
H
I
J
K
L
Coyote Creek

Figure 1-1

PREVIOUS STUDIES

Over the years, the hydrology of Coyote Creek has been analyzed and reanalyzed by both the District and the Corps of Engineers.

The following is a brief discussion of these studies:

Corps of Engineers Study

In February 1970, the Corps of Engineers issued a report, "Floodplain Information, Coyote Creek". The hydrologic results in that report reflect the data until that date. In June 1977, the San Francisco District of the Corps issued a preliminary office report, "Hydrologic Engineering Office Report, Guadalupe River and Coyote Creek". The results from the 1977 report are listed in Table 2 for comparison with District values.

Santa Clara Valley Water District

In 1961, and before the development of the regression equations for predicting the flood peaks and volumes, an attempt was made to define the design flood on Coyote by utilizing the unit hydrograph theory and the S-curve techniques of the Corps of Engineers. This work was conducted by Dale Burnett of the Corps and yielded standard project flood peaks that were the basis of the Corps of Engineers 1970 values.

In 1968-69, the results from the earliest version of regional regression equations were used to balance peaks and volumes of a model hydrograph. A 100-year flood inflow to Coyote was found to be 28,500 cfs, with a 23,000 cfs outflow and 29,300 cfs inflow to Anderson, with a 22,300 cfs outflow.

In late 1969, a further study was conducted utilizing the operation of the reservoir to give a firm yield of 34 cfs. The outflow from Anderson was estimated at 20,000 cfs and the rest of the design flow is listed in Table 2 (Comparison of Results and Choice of Storm Duration).

DISCUSSION OF PROCEDURES OF PRESENT STUDY

Basin Characteristics and 1% Regional Flows

The Coyote Creek Watershed is divided into two major watersheds, Coyote rural, which includes subareas A, B, C, D, E, F, G and H_1; and Coyote urban, which includes subareas H_2, I, J, K and L (Figure 1-1). The rural subareas have the following basin characteristics and the preliminary 1% flows as calculated directly by the regional equations.

Location	Sub-Area	Drainage Area (sq. mi.)	Mean Annual Precipitation (in.)	LL_s $(mi.^2/ft/ft^{1/2})$	Tc (Hours)	Q 1% (cfs)
Coyote Creek at USGA #1698	A	109.00	22.5	1935	6.0	21,848
Coyote Creek at Coyote Resv.	B	9.68	20.3	21	1.5	2,426
Coyote Creek at Anderson Resv.	C	51.84	20.5	1587	5.8	12,754
Coyote Creek Upstream Fisher Creek	D	12.55	19.5	853	4.8	4,465
Fisher Creek Upstream Coyote Creek	E	15.78	20.8	462	4.2	4,857
Coyote Creek at SCVWD #58	F	8.00	15.6	213	2.9	2,702
Upper Silver Creek Upstream Coyote Creek	G	5.96	15.4	275	2.35	2,290
Coyote Creek Upstream Silver Creek	H_1	2.18	14.8	12	.58	768

Urban areas, shown below, are treated with the District's method with a 60% ratio of imperviousness representing medium residential density.

Location	Sub-Area	* Drainage Area (sq. mi.)	Mean Annual Precipitation (in.)	Tc (Hours)
Coyote Creek Upstream Silver Creek	H2	Ai = .83 Ap = .56	14.8	Ti = .62 Tp = 2.62
Coyote Creek at Highway 280	I	Ai = 4.73 Ap = 3.15	14.0	Ti = .62 Tp = 2.62
Coyote Creek Upstream Lower Silver Creek	J	Ai = .83 Ap = .55	13.3	Ti = .62 Tp = 2.62
Coyote Creek at Montague Expwy	K	Ai = 2.44 Ap = 1.63	13.5	Ti = .62 Tp = 2.62
Coyote Creek at Highway 237	L	Ai = 1.38 Ap = .92	13.0	Ti = .62 Tp = 2.62

* Ai = Impervious urban area
Ap = Pervious urban area

In rural areas, the preliminary evaluation of design flows from regional regression equations is determined using the drainage areas, mean annual precipitation and basin factors. These preliminary values are compared with peak flows obtained from rainfall by employing the unit hydrograph theory and HEC-1 program. Although this latter technique is used mainly to define design hydrographs, the resultant peak flow is compared and balanced against the regional one. As will be shown later, the end result of the above procedure is further compared with results from the site-specific analysis of recorded data if available.

Urban areas are treated based on the District's method of urban hydrology. The density of land use is determined by the ratio of impervious to pervious parts.

Recorded Data and Application of Regional Equations

Inherent in the District hydrology method are checking procedures that use additional data wherever available. Recorded streamflow data, stochastically generated data, and data obtained from rainfall are utilized. Following is a discussion of these procedures and their application to Coyote Creek.

Streamflow Data Analysis

The basin has four unregulated streamflow stations, namely:

1. U.S.G.S. No. 1698, Coyote near Gilroy (15 years of record).
2. S.C.V.W.D. No. 57, Packwood Creek (15 years of record).
3. S.C.V.W.D. No. 10, Fisher Creek (33 years of record).
4. S.C.V.W.D. No. 56, Las Animas at O'Connell (14 years of record).

As will be shown later, these stations were used for evaluation of regional results. The data from these stations necessitated a site-specific adjustment for the District's new regional regression equation results.

There are also three streamflow stations that are biased by regulation and urbanization. These stations are:

1. S.C.V.W.D. No. 58, Coyote at Edenvale (59 years, 25 years after reservoirs were built).
2. U.S.G.S. No. 17000, Coyote at Madrone downstream of Anderson Reservoir.
3. S.C.V.W.D. No. 12, Coyote downstream of Coyote Reservoir and has intermittent reach.

Table 1 presents the 1% flow values as obtained from the regional and the statistical analysis at the locations of the above mentioned stations (excluding Station S.C.V.W.D. No. 12). Coyote near Gilroy gage analysis matched regional results for peak flows with no corrections. Yet, Packwood differed by as much as 21%, Madrone by as much as 10%, and Edenvale by as much as 20%. Fisher Creek matched because of the routing operation through the flat channel (not floodplain).

Without routing, Fisher Station differed by as much as 20%. If low 1976 data is excluded from the analysis, Las Animas also differed by as much as 20%. An in-depth analysis of the reliability of the data from each of those stations is shown below:

Coyote near Gilroy: U. S. G. S. #1698

The 15 years of data from U.S.G.S. #1698 gave a 1% flow estimate of 37,459 cfs, while a more realistic analysis of these data, after correlating it with 10 other stations around the area with more record, gave a 1% flow of 22,418 cfs. This station is considered good.

Packwood: The station was intended to measure flow flows. Out of the 15 years of peak flows, five years (1962, 1963, 1965, 1967, and 1968) were estimated. The peak flows estimated at these times were never checked against reliable rating curves. This station does not correlate with other stations in the area. Therefore, the data are considered unfit for complete statistical analysis.

Las Animas: Although this station was intended to measure low flows, the data showed better consistency than Packwood. Only one year (1976) was estimated. The statistical 1% value (8913 cfs) shown in Table 1-1 is based on data thorugh 1976. The 1976 maximum flow is equal to 1 cfs. This low value did not quality as a low outlier per the W.R.C. criteria; yet, alternate analysis of data without the 1976 value showed a more realistic result of 3990 cfs, which was used for site-specific adjustment.

Coyote at Madrone: The recording of data from this station started in 1903. The peak flow during that 1903 year (15,000 cfs) was first reported after 1967. Therefore, the reliability of this data point is in question. The data of subsequent years (prior to building Coyote and Anderson in 1936 and 1950, respectively) were reviewed and adjusted by the U.S.G.S. after 1967. This much delayed review casts a doubt about the reliability of the data. Also, the peak flows ranged from 8 cfs in 1924 to 25,000 cfs in 1911. An analysis of such a wide range of data per the W.R.C. proved that the 8 cfs is a low outlier; hence, the 1% flow was found to be 30,000 cfs. Yet, if this value (8 cfs) were actually 10 cfs, not a low outlier, the 1% value jumps to 55,391 cfs. This indicates the excessive sensitivity of the statistical procedure to low values, underscoring the need to carefully evaluate these data. A combination of those assumptions and doubtful inferences reduced the need to use this station for weighing peak flows.

Fisher Creek (S.C. #10): Although high peak flows could be affected by backwater from Coyote, few recorded values reflect this condition. The data were considered fit for a complete statistical analysis. The 1% flood was estimated at 4021 cfs while a more realistic value of 3941 cfs was obtained after correlation.

Coyote at Edenvale: This station is located below a floodplain that could accommodate as much as 10,000 acre-feet of storage for high flows above 10,000 cfs. The high flows therefore could be biased by extensive regulation even without reservoirs.

TABLE 1-1

COMPARISON OF STATISTICAL AND REGIONAL RESULTS

LOCATION	Peak, 1% Regional Ne	Peak, 1% Stat. N	Weighing Factor	1%, 24-Hr. Regional Ne	1%, 24-Hr. Stat. N	Weighing Factor	1%, 48-Hr. Regional Ne	1%, 48-Hr. Stat. N	Weighing Factor	1%, 72-Hr. Regional Ne	1%, 72-Hr. Stat. N	Weighing Factor	REMARKS
Coyote @ Gilroy	$\frac{21460}{19}$	$\frac{22420}{52}$	1.00	$\frac{12303}{15}$	$\frac{13000}{52}$	1.00	-	-	-	$\frac{6000}{15}$	$\frac{6530}{52}$	1.00	
Packwood Creek	$\frac{3505}{15}$	$\frac{2042}{15}$	0.79	$\frac{1350}{15}$	-	-	-	-	-	$\frac{1789}{15}$	-	-	Bad Data
Los Animas Creek	$\frac{9289}{19}$	$\frac{8913^*}{14}$	1.00(0.80)	-	-	-	-	-	-	-	-	-	-
Coyote @ Madrone	$\frac{36027}{19}$	$\frac{30000}{28}$	0.90	-	-	-	-	-	-	$\frac{10116}{15}$	$\frac{10000}{28}$	0.90	Data?
Fisher Creek	$\frac{3993}{19}$	$\frac{3941^{**}}{56}$	1.00(.80)	$\frac{1853}{19}$	$\frac{1802}{57}$	100	-	$\frac{1480}{57}$	-	$\frac{802}{15}$	$\frac{1326}{57}$	-	
Coyote @ Edenvale	$\frac{36663}{19}$	$\frac{20000}{17}$	0.80	-	-	-	-	-	-	-	-	-	Bad Data

* Considering the data until 1976. In 1976 maximum value of 1 cfs was recorded. This value is not a low outline per the W.R.C. Without this one data point the 1% value drops to 3,990 cfs, requiring a site-specific adjustment of 80% to the regional values.

** The 3,941 cfs value is the result of routing and combining regional flows of separate subareas through the channel. Without routing, the regional values require a site specific adjustment of 80%.

Preliminary Conclusions

Because of the above-mentioned data problems, it was decided that the three stations: Coyote near Gilroy, representing upper Coyote; Las Animas representing Anderson's local watershed, and Fisher, representing the lower areas, be used for calibration of regional results. The values obtained from the regional analysis match results of recorded data from the first station with no site-specific correction, while a site-specific adjustment is needed on the lower parts of the watershed.

Calculation of Adjustment Factors

The following are the mathematics that resulted in the Las Animas adjustment factor. These factors should be used in areas in the east side of Santa Clara Valley.

1% Flood

- from recorded data, SCVWD #56, Las Animas at O'Connell Ranch, 1963-1975:

  $m = 2.48000$ $\sigma = .607$

  $Q_{1\%} = 3990$ $Q_{10\%} = 1569$

- from 1976 Regional Equations:

  $m = 2.8269$ $\sigma = .607$

  $Ne_m = 8.35$ years $Ne_\sigma = 43.6$

  $Q_{1\%} = 9293$ $Q_{10\%} = 3593$

  $$Ne = \frac{\sigma^2}{E^2}$$

  $$= \frac{.607^2}{.128^2} = 22.5 \qquad Ne = 22.5$$

- $\text{weighted } 1\% = \dfrac{9293 \times 22.5 + 3990 \times 13}{35.5} = 7351$ cfs
- $1\% \text{ adjustment factor} = \dfrac{7351}{9293} = 0.79$, say $\underline{\underline{80\%}}$
- $\text{weighted } 10\% = \dfrac{3593 \times 22.5 + 1569 \times 13}{35.5} = 2852$
- $10\% \text{ adjustment} = \dfrac{2852}{3593} = 0.794$, say $\underline{\underline{80\%}}$

Similarly, 19%, 7%, and 7% adjustments are calculated for the 24-hour, 48-hour, and 72-hour, respectively. Adjustments for the 10% regional flood flows are 20, 18, 10, and 10% for the peak, 24-hour, 48-hour, and 72-hour, respectively.

Precipitation Data

The basin is instrumented by 13 precipitation stations, namely:

1. S.C. No. 98, Shanti Ashram
2. S.C. No. 18, Coit Ranch
3. S.C. No. 29, Gilroy 8NE (U.S.W.B.)
4. S.C. No. 75, Peabody Ranch
5. S.C. No. 17, Coe Park
6. S.C. No. 102, UTC
7. S.C. No. 65, Mt. Hamilton (U.S.W.B.)
8. S.C. No. 67, Mt. Hamilton (U.S.W.B.)
9. S.C. No. 34, Hasking Ranch
10. S.C. No. 99, Penitencia Water Treatment Plant
11. S.C. No. 86, San Jose
12. S.C. No. 23, Curtner Ranch
13. S.C. No. 41, Leroy Anderson

The data from these stations were used in the determination and transposition of the storm over the basin.

Storm Duration

It was found that, on basins that do not have significant* reservoir volumes, the 24-hour storm yields acceptable flood hydrographs. Because the Coyote Basin has two large reservoirs, two storms, the 24-hour and the 72-hour (obtained from above stations) were analyzed. The results shown in Table 1-2 proved that with minimum regulation (initial storages are full), the 1% flood obtained from a balanced 24-hour storm is more critical (i.e., higher design peak outflows). Yet, when more initial flood control storage becomes available, the 72-hour storm as balanced with regional peak, 24-hour, 48-hour, and 72-hour volumes becomes more critical. In the case of 16,000 acre-feet and 81,000 acre-feet of initial storage in Coyote and Anderson (as adopted by the Corps of Engineers), it was found that the 72-hour storm yields higher flows from Anderson than the 24-hour one. Although the results of routing the 24-hour event were equal to the Corps' 1976 results, the 72-hour event (a more critical event) was considered for further analysis. The 72-hour storm was obtained by the statistical analysis of data from the previously mentioned 13 precipitation stations per the Department of Water Resources frequency analysis studies. The transposition of the storm was assumed to follow the pattern of the mean annual precipitation.

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*Significant means that the available storage for flood control is bigger than the part of the hydrograph volume preceding the peak.

Temporal and Spacial Storm Distribution

Time distribution of the above-discussed 72-hour design storm could occur in many forms. The following four distributions were analyzed and results are shown in Table 1-3:

1. The Corps of Engineer's Standard Storm Distribution for Northern California.

2. Corps of Engineers' Standard Storm for Southern California.

3. 1955 Storm from Stayton Mine, Freedom and Hollister (1977 Corps of Engineers Draft Report - Hydraulics of Guadalupe River and Adjacent Streams, Plate 18).

4. 1958 Storm over the County as distributed from seven precipitation stations.

TABLE 1-2

COYOTE CREEK HYDROLOGY
1% FLOOD
CHOICE OF STORM DURATION

	Location	Area Sq Mi	24 Hrs Storm, Peak & 24 Hr Flood				72 Hrs Storm, Peak 24 & 72 Hr Flood					Statistics		C of Eng Values (Future cond'ts	Previous Design Flow (Green book)	C of Eng Los Osos[1/]
			Full Resvrs	Coyote =16000 Andr= 81000 AF	Coyote= 16000 Ander= 81000 Los Osos =0/30000	S.C. Initials Coyote= 16,000 And.= 70400	Full R'srs	Coyote =16000 Andrs= 81000 AF	As Before +Los Osos Empty	S.C. Initials Coyote =16000 Andr= 70400	Balance Peak 24, 48,&72 Coyote =16000 And= 81000	Recorded Data	Recorded & Recon-stituted Data			
1)	At USGS # 1698	109	21848	21848	21848	21848	21848	21848	21848	21848	21848	37459	22418	-	25300	
2)	Los Osos Outflow	109	21848	21848	2123	21848	21848	21848	2767	21848	21848			-		
3)	Coyote Inflow	118.68	21920	21920	3286	21920	21436	21436	4014	21460	21460			20000	28200	
4)	Coyote Outflow	118.68	17449	14973	432	14973	16820	16643	1720	16706	16706	7074[2/]			20700	
5)	Anderson Inflow	193	29647	23329	17839	23329	29897	29388	16970	29499	29499			21000	29970	
6)	Anderson Outflow	193	23555	15298	5730	8124	23923	22103	8846	18048	22343			14600	20000	2700
7)	U/S of Fisher	205.27	23271	14645	5666	7889	24197	21959	9453	17391	22230			-		
8)	Fisher U/S Coyote	15.78	3911	3911	3911	3911	3993	3993	3993	3993	3993	4021	3941	-		
9)	Coyote D/S Fisher	221.05	24319	15009	8468	8454	25493	23055	11172	17692	23324			15000	19400	3500
10)	@ S.C. Sta. #58	229.05	24563	14946	9554	9540	25906	23133	11714	17565	23454	28929[3/]		-		
11)	U/S of Upper Silver	232.62	24703	14950	9633	9619	26066	23237	11995	17560	23457			14800	18000	3800
12)	U. Silver U/S Coyote	5.96	2290	2290	2290	2290	2290	2290	2290	2290	2290			-		
13)	Coyote D/S Silver	238.55	24753	14968	10965	10951	26196	23330	13012	17564	23504			14800		3800
14)	At 280	246.46	25452	14857	11337	11317	26728	23286	13204	17415	23500					
15)	U/S of Silver-Thompson	247.84	25571	14845	11427	11401	26734	23289	13339	17360	23504					
16)	Silver-Thompson										Balancing w/48 Hr over & above the 24&72 Hrs does not add information					
17)	D/S Silver-Thompson													14200		6500
18)	Penitencia															
19)	D/S Penitencia													14200		9000
20)	@ Trimble															
21)	@ 237															

1/ Represent the Corps of Engineers values, w/Los Osos Reservoir, w/30,000 Ac. Ft. Flood Control.
2/ These values represent regulated flows. They are used for reference only.
3/ Value inappropriately includes all records from 1917 do date. Coyote was built in 1935, Anderson 1950. Analysis of regulated data after 1950 gave a 1% value equal to = 5054 cfs which has been exceeded at least once in 25 years.
4/ Los Animas adjustment is made by analyzing the recorded data at Los Animas S.C.#56. The adjustment factors were 20, 19, 7, & 7% for the peak, 24, 48, & 72-hour volumes, respectively. These factors were applied to Area C3 only. This is the recommended final value.

TABLE 1-3

COYOTE CREEK
1% DESIGN FLOOD DETERMINATION
CHOICE OF TIME DISTRIBUTION

LOCATION	DRAINAGE AREA SQ. MI.	CORPS' 1977 Storage Coyote=16,000 Anderson=81,000 RANDOM STORM Exist.	W/Los Osos	DIST. #1 Coyote=16,000 Anderson=70,400 N. CAL. STAN. ST. Exist.	W/Los Osos	DIST. #2 Coyote=16,000 Anderson=76,000 S. CAL STAN. ST. Exist.	W/Los Osos	DIST. #3 Coyote=16,000 Anderson=76,000 1977 COE STAN. ST. Exist.	W/Los Osos	DIST. #4 Coyote=16,000 Anderson=76,000 1977 COE STAN. ST. Exist.	W/Los Osos	DIST. #5 Coyote=16,000 Anderson=76,000 1958 STORM Exist.	W/Los Osos	FINAL CHOICE Coyote=16,000 Anders=81,000 COE PL18 Exist.	COE PL18 District W/L.Osos
Coyote - Inflow	119	20,000	-	21,460	4,025	21,790	3,909	21,658	3,082	21,658	3,082	23,495		22,000	3,082
Coyote - Outflow	119	-	-	16,706	2,190	16,330	2,186	13,242	2,130	13,242	2,130	19,160		13,240	2,130
Anderson-Inf.	193	21,000	-	26,948	14,750	26,523	15,100	19,968	15,040	19,968	15,040	28,880		20,000	15,040
Anderson-Outf	193	14,600	2,700	16,858	2,100	14,640	2,850	12,615	3,000	12,615	2,985	19,850		15,000	2,985
D/S Fisher	221	15,000	3,500	16,539	8,900	14,240	7,900	13,300	7,900	13,495	7,940	20,500		15,000	7,940
D/S U.Silver	239	14,800	3,800	16,452	11,000	14,100	9,980	13,560	9,660	12,489	7,400	20,000		14,500	7,400
D/S L.Silver	291	14,200	9,000	19,000	16,100	16,600	15,100	16,440	14,820	14,500	12,000	20,700		14,500	12,000
D/S Upper Penitencia	315	14,200	13,000	20,000	20,000	19,000	19,000	19,400	19,400	14,500	14,500	20,600		14,500	14,500
		● Arbitrary choice of Storm Distribution ● Arbitrary Centering of Storm, transposing storm according to areas. ● Excessive storage in Edenvale area.		● Los Animas correct on Area C_3 only. ● Muskingum channel routing with x = 0 is consistently used on all branches.		● As before w/L.A. adj. on low lands.		● As before w/C.E. Stand. Storm Pl 18. ● Silver Diversion to contribute 950 only.		● As before. ● Puls channel routing was used on Creek D/S Anderson. S-D curve obtained from COE Pl 17. ● Muskingum routing on tributaries.		● As before with 1958 Storm Distribution from 7 local stations.		● Based on Corps' Study.	● Based on District #4 Study.

From Table 1-2 it is obvious that the 72-hour storm becomes more critical than shorter storms as more volume becomes available, therefore, the 72-hour storm was chosen as a design storm.

The space distribution was made comparable with the variation inherent in the mean annual precipitation isolines and the thirteen stations.

Five temporal distributions were considered. Table 1-3 shows the comparative results of these distributions, together with the choice of the final alternative and the final design flows. The footnotes in that table discuss the underlying assumptions taken in each alternative. It is noticed that the storm distribution could have a major effect on the flows downstream of Anderson. Yet, the single most important factor is the initial available storage for flood control. The following section discusses the initial storage determinations.

Determination of Initial Storage Volumes

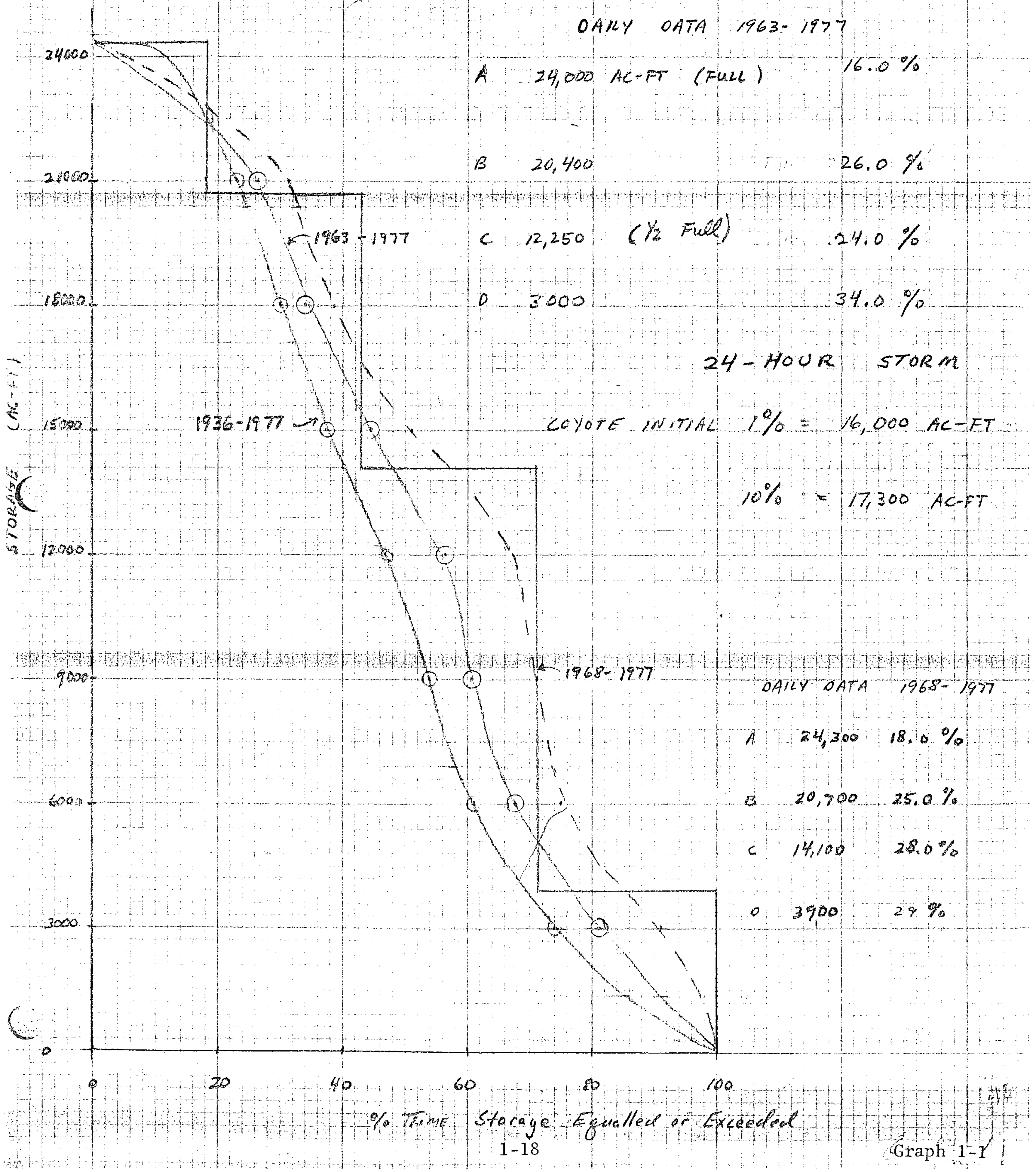
The best way to determine the initial storage for a certain reservoir is to have optimum criteria and rule curves for the operation of that reservoir or reservoirs.

Although the basic intention of building Coyote and Anderson Reservoirs was water conservation, flood control benefit accrued incidentally. Over the years, this incidental availability of storage for flood control became obvious. In the absence of a qualified and quantified operational study, it was decided that the historical reservoir operation should be evaluated and trends should be pointed out. The clarification of these trends is important because of state and federal restrictions that were placed on the operations of the reservoirs. An example was the 1962 letter from the Bureau of Reclamation to District Chief Engineer recommending that the minimum summer pool volume at Anderson Reservoir be at least 20,000 acre-feet. Also recognized was the

P-1 SANTA CLARA VALLEY WATER DISTRICT, San Jose, California

By M.K. Date 6-77 Chkd. By Date Sheet No. 1 of 2

Subject COYOTE RESERVOIR STORAGE-DURATION CURVE Job No.



DAILY DATA 1963-1977

A	24,000 AC-FT	(FULL)	16.0 %
B	20,400		26.0 %
C	12,250	(½ Full)	24.0 %
D	3000		34.0 %

24-HOUR STORM

COYOTE INITIAL 1% = 16,000 AC-FT

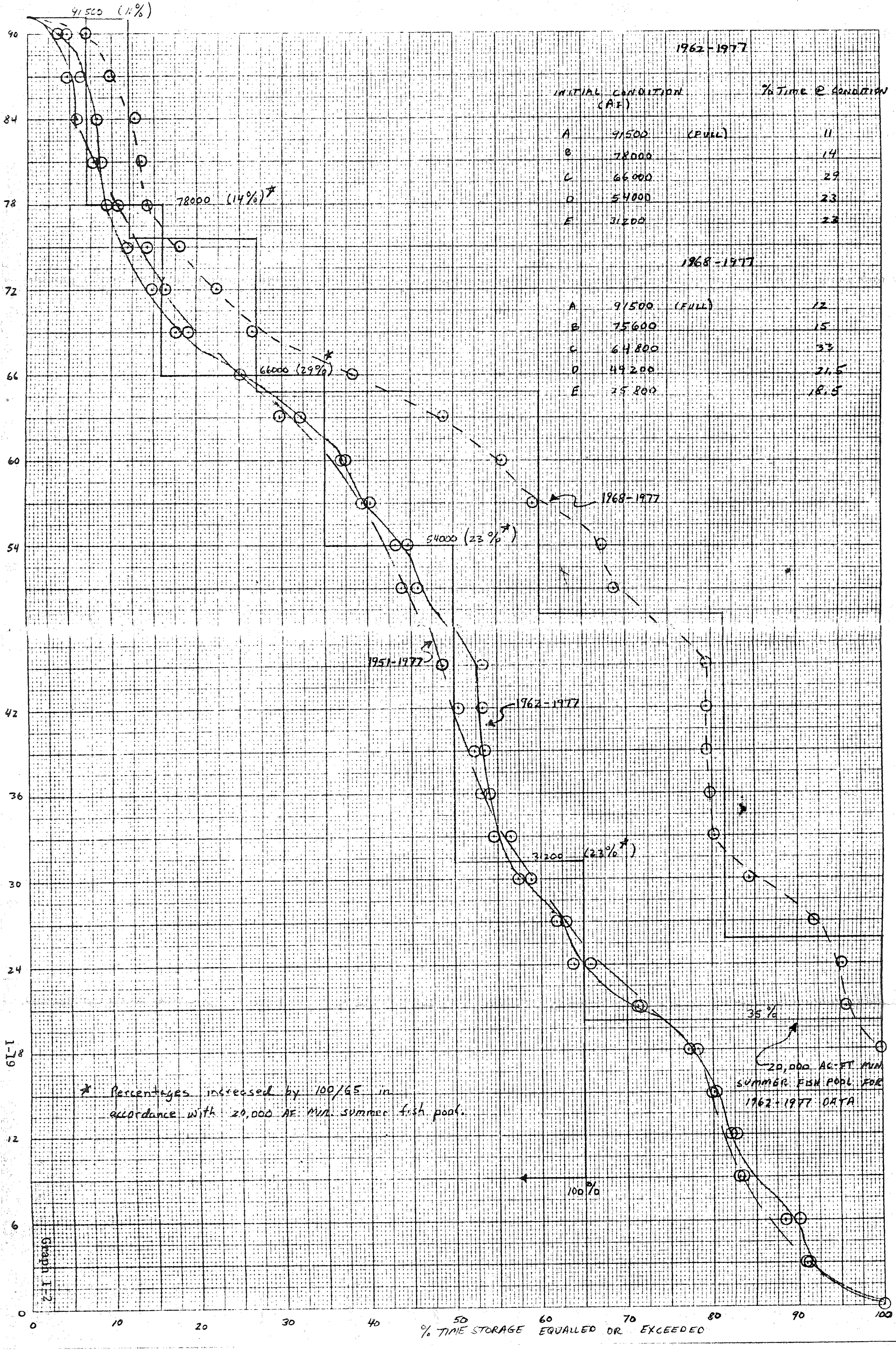
10% = 17,300 AC-FT

DAILY DATA 1968-1977

A	24,300	18.0 %
B	20,700	25.0 %
C	14,100	28.0 %
D	3900	29 %

Graph 1-1

ANDERSON RESERVOIR STORAGE-DURATION
DAILY DATA 1951-1977



1962-1977

	INITIAL CONDITION (AF)		% TIME @ CONDITION
A	91500	(FULL)	11
B	78000		14
C	66000		29
D	54000		23
E	31200		23

1968-1977

A	91500	(FULL)	12
B	75600		15
C	64800		33
D	49200		21.5
E	35800		18.5

* Percentages increased by 100/65 in accordance with 20,000 AF min. summer fish pool.

Graph 1-2

need for recreation, which resulted in the reservoirs remaining fuller more often after 1968. Graphs 1-1 and 1-2 show the analyses of the storage-frequency-duration of both Coyote and Anderson Reservoirs. It is clear from these curves that there is a continual trend toward keeping the reservoirs nearer to capacity. By routing various floods through reservoirs at various initial storages, a compounded probability curve was constructed and the incidental initial storages for various frequencies were determined. During this search, it was found that for Anderson results obtained from the curve based on 1968-77 data are equal to the results obtained from the 1962-77 data with a minimum pool of 20,000 acre-feet.

The curve which includes the combined storage of Coyote and Anderson is not shown. It yielded comparable results.

Table 1-4 is a summary of initial storages found by the various assumptions. The recommended storages found to be used with design flows could be based on the 1963-77 data but, because of the huge initial storage available at Anderson and because of the trend toward keeping reservoirs fuller, the Corps of Engineers' storage (81,000 acre-feet) was chosen.

Los Osos

A future reservoir of 30,000 acre-feet storage, 3,000 cfs outlet works capacity and a 150 spillway length was assumed existing on Coyote Creek downstream of Canada De Los Osos.

This reservoir is assumed to be operated for flood control alone; i.e., totally empty all the wet season. The effect of this structure on the 1% flood is shown in Table 1-3. Although the flood reduction is noticeable upstream of Silver-Thompson confluence, the reduction becomes insignificant downstream of that point.

An important hydrologic aspect of the new reservoir is that it delays the last, critical hump of the generated hydrograph to such an extent that the hump no longer remains critical. Successive humps become more critical.

TABLE 1-4

ANDERSON AND COYOTE RESERVOIRS INITIAL STORAGE (AF)

1% and (10%)

Daily Data	24-Hour Storm				72-Hour Storm	
	Coyote	Anderson			Coyote	Anderson
1963-1977*	16,000 (17,300)	68,000 (69,000)			16,000 (16,500)	70,400 (72,400)
1963-1977* Combined Anderson and Coyote, Coyote at 1% = 16,000, 10% = 16,500					16,000 (16,500)	70,700 (76,000)
1968-1977	16,600 (18,000)	71,200 (73,400)			16,000 (16,500)	69,600 (72,000)
1963-1977*					16,000 (16,500)	70,400 (72,400)
Corps of Engineers**					16,000	81,000

* Values obtained assuming minimum summer fish pool of 20,000 AF in Anderson Reservoir. Analysis of daily data between 1963-1977 without a minimum 20,000 AF pool led to unreliable results because of the inability to peak storage volumes from routed flows in Anderson Reservoir when it was at low levels.

** It has been noticed that the Corps' initial storage values are higher than the coincidental. Because of the previously mentioned trend of fuller reservoirs, those values are chosen for design purposes.

Analysis of Results

Tables 1-2 and 1-3 summarize the work. Table 1-2 shows the Corps of Engineers' values together with steps needed to choose the 72-hour storm as the critical duration. Based on that duration, Table 1-3 shows the various runs used to determine the storm distribution and other assumptions. Although the District's values at locations upstream of Coyote Reservoir are equal to the Corps', the values downstream of the reservoirs are either higher than the Corps' (District #5) or slightly lower. Because of this, and after the August 5, 1977 meeting at Nolte's offices, the values adopted as design flows for the Coyote Creek system are the ones that yield similar flows with the Corps'. These values are shown in Table 1-3 (District #6).

For the determination of Los Osos' effect on the Coyote System, the Corps' values appear to be unrealistically low. Therefore, the District's values, as described in Alternative District #4 (Table 1-3), should be used.

Summary

The following are the various conclusions that could be obtained from Tables 1-2 and 1-3.

1. It was found from Table 1-2 that the flood which is based on a 24-hour storm yielded a slightly higher peak inflow to Coyote than the 72-hour storm. The outflow, though, is insignificantly different if the reservoirs are full.

2. If the reservoirs are at an initial storage as computed by the Corps of Engineers, then the flood which is based on the 72-hour storm yields larger peak outflow than the 24-hour storm (Table 1-2).

3. It was found that the Corps of Engineers' values are identical to our values for the 24-hour storm if the initial storages are 16,000 and 81,000 acre-feet for Coyote and Anderson (Table 1-2).

4. It was found that the introduction of Los Osos Reservoir (30,000 acre-feet capacity; 3,000 cfs maximum outlet flow and 150 feet of spillway) does not reduce the flows in the lower reaches by as much as the Corps of Engineers indicate in their report (outflow from Anderson is about 2,700 cfs).

5. The regional values check closely with recorded data at the U.S.G.S. Station No. 1698 (upstream of Coyote Reservoir) for the 24-hour peaks and 72-hour volumes.

6. The initial storage at Anderson is the most critical input in the determination of the downstream design flood flows. In the absence of an optimum operational criterion our initial storage is based on the availability of initial storage at Anderson (approximately 91,000 acre-feet) and a continued trend toward water conservation, the Corps' initial storage of 81,000 acre-feet was chosen. Under these less-than-full conditions, the hydrograph downstream of Anderson acquires two peaks. The first is the one from the upper watershed and the second from the lower areas. The second peak could become more critical as the flood wave travels downstream, especially when enough storage becomes available in the upper reaches.

7. The 72-hour storm when balanced with peaks, 24, 48, and 72-hour volumes yields the same values as when balanced with only the peak 24 and 72-hour volumes. This is conducive to solving the problems associated with a proper storm distribution.

8. The design flows are obtained from the condition that includes the most information and has a duration that yields the most critical design flows.

9. This condition is based on the 72-hour storm and the Corps of Engineers' storage at Anderson.

10. The summary of the recommended 1% and 10% design flows for Coyote Creek without Los Osos Reservoir is given in Table 1-5 and 1-6. Table 1-7 lists the recommended 1% design flows from Coyote Creek with Los Osos Reservoir and the District's storage at Anderson.

The values in those tables should be rounded up to significant figures.

COYOTE CREEK
Recommended 1% Design Flows
Without Los Osos Reservoir

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	10064.	7688.	4733.	2193.	41.00	East Fork Coyote Cr. u/s Middle Fork
ROUTED TO	1	10064.	7688.	4733.	2193.	41.00	
HYDROGRAPH AT	1	6589.	5075.	2858.	1262.	22.00	
ROUTED TO	1	6589.	5075.	2858.	1262.	22.00	Middle Fork Coyote Cr. u/s East Fork
2 COMBINED	1	16653.	12749.	7591.	3455.	63.00	
ROUTED TO	11	14153.	12279.	7481.	3446.	63.00	
HYDROGRAPH AT	11	9881.	7259.	4578.	2080.	46.00	Upper Coyote Cr. u/s East Fork
ROUTED TO	11	9881.	7259.	4578.	2080.	46.00	
2 COMBINED	11	21848.	18361.	11389.	5652.	109.00	Upper Coyote Cr @ USGS # 1698
ROUTED TO	2	20400.	18172.	11304.	5645.	109.00	
HYDROGRAPH AT	2	2426.	1557.	958.	380.	9.68	
ROUTED TO	2	2426.	1557.	958.	380.	9.68	
2 COMBINED	2	21658.	19423.	12209.	6024.	118.68	Inflow to Coyote Reservoir
ROUTED TO	2	13242.	12794.	9292.	4584.	118.68	Outflow from Coyote Reservoir
ROUTED TO	3	13064.	12629.	9236.	4583.	118.68	
HYDROGRAPH AT	3	4147.	2911.	1624.	668.	12.10	
ROUTED TO	3	4147.	2911.	1624.	668.	12.10	
2 COMBINED	3	14025.	13589.	10031.	5182.	130.78	
HYDROGRAPH AT	31	3505.	2368.	1350.	548.	10.10	Packwood Creek @ SCVWD #57
ROUTED TO	3	3311.	2368.	1348.	548.	10.10	
HYDROGRAPH AT	3	10203.	7776.	4725.	2551.	51.84	
ROUTED TO	3	10203.	7776.	4725.	2551.	51.84	
3 COMBINED	3	19968.	19279.	15407.	8136.	192.72	Inflow to Anderson Reservoir
ROUTED TO	3	15000.	14200.	11648.	6440.	192.72	Outflow from Anderson Reservoir
ROUTED TO	4	15000.	14056.	11544.	6422.	192.72	
HYDROGRAPH AT	4	3572.	2471.	1344.	674.	12.55	
ROUTED TO	4	3572.	2471.	1344.	674.	12.55	
2 COMBINED	4	15000.	14697.	12084.	6862.	205.27	Coyote Creek u/s Fisher Cr.
HYDROGRAPH AT	41	2100.	1227.	770.	313.	6.70	Upper Fisher Creek @ Kalana Ave.
ROUTED TO	42	1603.	1224.	763.	312.	6.70	
HYDROGRAPH AT	42	2267.	1402.	820.	323.	6.68	Middle Fisher Creek @ Bailey St.
ROUTED TO	42	2267.	1402.	820.	323.	6.68	
2 COMBINED	42	3846.	2604.	1581.	635.	13.38	
ROUTED TO	4	3406.	2606.	1576.	635.	13.38	
HYDROGRAPH AT	4	934.	546.	294.	119.	2.40	
ROUTED TO	4	934.	546.	294.	119.	2.40	
2 COMBINED	4	3834.	3098.	1861.	753.	15.78	Fisher Creek U/s Coyote Cr.
2 COMBINED	4	15000.	15000.	12659.	7546.	221.05	Coyote Creek D/s Fisher Cr.
ROUTED TO	5	15000.	15000.	12415.	7490.	221.05	
HYDROGRAPH AT	5	2162.	1283.	748.	361.	8.00	
ROUTED TO	5	2162.	1283.	748.	361.	8.00	
2 COMBINED	5	15000.	15000.	12692.	7799.	229.05	Coyote Creek @ Edenvale (SCVWD #58)
ROUTED TO	6	15000.	15000.	12629.	7780.	229.05	
HYDROGRAPH AT	6	614.	355.	186.	79.	2.18	
ROUTED TO	6	614.	355.	186.	79.	2.18	
HYDROGRAPH AT	6	197.	152.	82.	42.	0.83	
HYDROGRAPH AT	6	120.	92.	48.	22.	0.56	
2 COMBINED	6	312.	239.	130.	65.	1.39	
ROUTED TO	6	151.	151.	129.	65.	1.39	
2 COMBINED	6	762.	501.	315.	144.	3.57	
2 COMBINED	6	14500.	14500.	12699.	7881.	232.62	Coyote Creek u/s Silver Creek
HYDROGRAPH AT	6	1832.	1149.	600.	285.	5.96	
ROUTED TO	16	950.	950.	592.	284.	5.96	Upper Silver Creek u/s Coyote Cr.
ROUTED TO	6	950.	950.	592.	284.	5.96	
2 COMBINED	6	14500.	14500.	12874.	8116.	238.58	Coyote Creek D/s Silver Creek

```
ROUTED TO            7    14500.    14500.    12739.     8095.    238.58
HYDROGRAPH AT        7     1067.      820.      444.      228.      4.73
HYDROGRAPH AT        7      638.      491.      254.      120.      3.15
 2 COMBINED          7     1679.     1284.      697.      348.      7.88
ROUTED TO            7      850.      844.      696.      348.      7.88
 2 COMBINED          7    14500.    14500.    12875.     8317.    246.46  Coyote Creek @ 280
ROUTED TO            8    14500.    14442.    12786.     8304.    246.46
HYDROGRAPH AT        8      178.      137.       74.       38.      0.83
HYDROGRAPH AT        8      106.       82.       43.       20.      0.55
 2 COMBINED          8      280.      214.      116.       58.      1.38
ROUTED TO            8      147.      146.      116.       58.      1.38
 2 COMBINED          8    14500.    14470.    12803.     8340.    247.84  Coyote Creek u/s Lower Silver Cr.
HYDROGRAPH AT        8     5477.     5187.     4041.     3304.     43.50  Lower Silver Creek u/s Coyote Cr.
 2 COMBINED          8    14493.    14349.    13151.     9900.    291.34  Coyote Creek d/s Lower Silver Cr.
HYDROGRAPH AT        8     4635.     3864.     2154.     1596.     23.91  Upper Penitencia Creek u/s coyote Cr.
 2 COMBINED          8    14550.    14349.    13293.    10625.    315.25  Coyote Creek d/s Upper Penitencia Cr.
ROUTED TO            9    14456.    14320.    13279.    10615.    315.25
HYDROGRAPH AT        9      531.      408.      221.      113.      2.44
HYDROGRAPH AT        9      319.      245.      127.       60.      1.63
 2 COMBINED          9      837.      640.      347.      173.      4.07
ROUTED TO            9      436.      433.      346.      173.      4.07
 2 COMBINED          9    14500.    14300.    13332.    10734.    319.32  Coyote Creek @ Montague Expressway
ROUTED TO           10    14500.    14300.    13331.    10729.    319.32
HYDROGRAPH AT       10      289.      222.      120.       62.      1.38
HYDROGRAPH AT       10      173.      133.       69.       33.      0.92
 2 COMBINED         10      455.      348.      189.       94.      2.30
ROUTED TO           10      245.      243.      189.       94.      2.30
 2 COMBINED         10    14500.    14300.    13442.    10791.    327.62  Coyote Creek @ 237
```

COYOTE CREEK
Recommended 10% Design Flows
Without Los Osos Reservoir

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	4258.	3697.	2075.	1003.	41.00	East Fork Coyote Cr. U/S Middle Fork
ROUTED TO	1	4258.	3697.	2075.	1003.	41.00	
HYDROGRAPH AT	1	2844.	2507.	1275.	601.	22.00	Middle Fork Coyote Cr. U/S East Fork
ROUTED TO	1	2844.	2507.	1275.	601.	22.00	
2 COMBINED	1	7098.	6189.	3350.	1604.	63.00	
ROUTED TO	11	6626.	5937.	3330.	1603.	63.00	
HYDROGRAPH AT	11	3987.	3284.	1904.	912.	46.00	Upper Coyote Cr. U/S East Fork
ROUTED TO	11	3987.	3284.	1904.	912.	46.00	
2 COMBINED	11	8665.	8161.	4711.	2382.	109.00	Upper Coyote Cr. @ USGS # 1698
ROUTED TO	2	8627.	8050.	4705.	2381.	109.00	
HYDROGRAPH AT	2	1111.	772.	442.	196.	9.68	
ROUTED TO	2	1111.	772.	442.	196.	9.68	
2 COMBINED	2	9252.	8672.	5134.	2576.	118.68	Inflow to Coyote Reservoir
ROUTED TO	2	3413.	3296.	2447.	1199.	118.68	Outflow from Coyote Reservoir
ROUTED TO	3	3358.	3262.	2438.	1199.	118.68	
HYDROGRAPH AT	3	1730.	1394.	697.	315.	12.10	
ROUTED TO	3	1730.	1394.	697.	315.	12.10	
2 COMBINED	3	3711.	3554.	2615.	1503.	130.78	
HYDROGRAPH AT	31	1507.	1160.	599.	269.	10.10	Packwood Creek @ SCVWD #57
ROUTED TO	3	1467.	1158.	599.	269.	10.10	
HYDROGRAPH AT	3	3910.	3437.	1917.	1016.	51.84	
ROUTED TO	3	3910.	3437.	1917.	1016.	51.84	
3 COMBINED	3	7001.	6267.	4984.	2789.	192.72	Inflow to Anderson Reservoir
ROUTED TO	3	548.	548.	548.	548.	192.72	Outflow from Anderson Reservoir
ROUTED TO	4	548.	548.	548.	548.	192.72	
HYDROGRAPH AT	4	1725.	1365.	666.	298.	12.55	
ROUTED TO	4	1725.	1365.	666.	298.	12.55	
2 COMBINED	4	2260.	1900.	1205.	839.	205.27	Coyote Creek U/S Fisher Cr.
HYDROGRAPH AT	41	950.	617.	360.	157.	6.70	Upper Fisher Creek @ Kalana Ave.
ROUTED TO	42	768.	615.	359.	157.	6.70	
HYDROGRAPH AT	42	983.	677.	362.	158.	6.68	Middle Fisher Creek @ Bailey St.
ROUTED TO	42	983.	677.	362.	158.	6.68	
2 COMBINED	42	1749.	1278.	720.	314.	13.38	
ROUTED TO	4	1600.	1279.	719.	314.	13.38	
HYDROGRAPH AT	4	419.	273.	133.	54.	2.40	
ROUTED TO	4	419.	273.	133.	54.	2.40	
2 COMBINED	4	1857.	1522.	850.	369.	15.78	Fisher Creek U/S Coyote Cr.
2 COMBINED	4	4078.	3421.	2054.	1207.	221.05	Coyote Creek D/S Fisher Cr.
ROUTED TO	5	4823.	2821.	1987.	1433.	221.05	
HYDROGRAPH AT	5	810.	544.	291.	137.	8.00	
ROUTED TO	5	810.	544.	291.	137.	8.00	
2 COMBINED	5	4824.	3170.	2271.	1568.	229.05	Coyote Creek @ Edenvale (SCVWD # 58)
ROUTED TO	6	3301.	3081.	2256.	1587.	229.05	
HYDROGRAPH AT	6	321.	201.	95.	38.	2.18	
ROUTED TO	6	321.	201.	95.	38.	2.18	
HYDROGRAPH AT	6	119.	90.	45.	21.	0.83	
HYDROGRAPH AT	6	69.	51.	22.	10.	0.56	
2 COMBINED	6	185.	137.	67.	30.	1.39	
ROUTED TO	6	141.	131.	67.	30.	1.39	
2 COMBINED	6	461.	329.	162.	68.	3.57	
2 COMBINED	6	3343.	3240.	2391.	1655.	232.62	Coyote Creek U/S Silver Cr.
HYDROGRAPH AT	6	674.	489.	230.	107.	5.96	
ROUTED TO	16	579.	465.	228.	106.	5.96	Upper Silver Creek U/S Coyote Cr.
2 COMBINED	6	3636.	3544.	2613.	1760.	238.58	Coyote Creek D/S Silver Creek

```
HYDROGRAPH AT       7     644.      484.      241.      111.      4.73
HYDROGRAPH AT       7     368.      272.      120.       51.      3.15
 2 COMBINED         7     995.      739.      360.      162.      7.88
ROUTED TO           7     794.      714.      359.      162.      7.88
 2 COMBINED         7    3781.     3675.     2886.     1911.    246.46  Coyote Creek @ 280
ROUTED TO           8    3656.     3573.     2858.     1901.    246.46
HYDROGRAPH AT       8     106.       80.       40.       18.      0.83
HYDROGRAPH AT       8      60.       40.       19.        8.      0.55
 2 COMBINED         8     163.      121.       59.       27.      1.38
ROUTED TO           8     138.      118.       59.       27.      1.38
 2 COMBINED         8    3695.     3616.     2903.     1928.    247.84  Coyote Creek u/s Lower Silver Cr.
HYDROGRAPH AT       8    3456.     3289.     2297.     1789.     43.50  Lower Silver Creek u/s Coyote Cr.
 2 COMBINED         8    6782.     6683.     5185.     2717.    291.34  Coyote Creek d/s Lower Silver Cr.
HYDROGRAPH AT       8    1715.     1526.      756.      257.     23.91  Upper Penitencia Creek u/s Coyote Cr.
 2 COMBINED         8    8160.     7886.     5939.     2974.    315.25  Coyote Creek d/s Upper Penitencia Cr.
ROUTED TO           9    7995.     7825.     5911.     2971.    315.25
HYDROGRAPH AT       9     322.      242.      121.       56.      2.44
HYDROGRAPH AT       9     184.      136.       60.       25.      1.63
 2 COMBINED         9     498.      370.      180.       81.      4.07
ROUTED TO           9     409.      359.      180.       81.      4.07
 2 COMBINED         9    8193.     7993.     6064.     3052.    319.32  Coyote Creek @ Montague Expressway
ROUTED TO          10    8163.     7977.     6047.     3051.    319.32
HYDROGRAPH AT      10     174.      131.       65.       30.      1.38
HYDROGRAPH AT      10     100.       74.       33.       14.      0.92
 2 COMBINED        10     270.      200.       98.       44.      2.30
ROUTED TO          10     231.      195.       97.       44.      2.30
 2 COMBINED        10    8267.     8068.     6129.     3094.    321.62  Coyote Creek @ 237
```

COYOTE CREEK

Recommended 1% Design Flows

With Los Osos Reservoir

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	10064.	7688.	4733.	2193.	41.00	East Fork Coyote Cr. u/s Middle Fork
ROUTED TO	1	10064.	7688.	4733.	2193.	41.00	
HYDROGRAPH AT	1	6589.	5075.	2858.	1262.	22.00	
ROUTED TO	1	6589.	5075.	2858.	1262.	22.00	Middle Fork Coyote Cr. u/s East Fork
2 COMBINED	1	16653.	12749.	7591.	3455.	63.00	
ROUTED TO	11	14153.	12279.	7481.	3446.	63.00	
HYDROGRAPH AT	11	9881.	7259.	4578.	2080.	46.00	Upper Coyote Cr. u/s East Fork
ROUTED TO	11	9881.	7259.	4578.	2080.	46.00	
2 COMBINED	11	21848.	18361.	11389.	5652.	109.00	Inflow to Los Osos Reservoir
ROUTED TO	11	2535.	2527.	2468.	2274.	109.00	Outflow from Los Osos Reservoir @
ROUTED TO	2	2533.	2526.	2468.	2274.	109.00	USGS #1698
HYDROGRAPH AT	2	2426.	1557.	958.	380.	9.68	
ROUTED TO	2	2426.	1557.	958.	380.	9.68	
2 COMBINED	2	3082.	2757.	2578.	2462.	118.68	Inflow to Coyote Reservoir
ROUTED TO	2	2130.	2126.	2075.	1267.	118.68	Outflow from Coyote Reservoir
ROUTED TO	3	2126.	2125.	2074.	1246.	118.68	
HYDROGRAPH AT	3	4147.	2911.	1624.	668.	12.10	
ROUTED TO	3	4147.	2911.	1624.	668.	12.10	
2 COMBINED	3	4549.	3314.	2106.	1376.	130.78	
HYDROGRAPH AT	31	3505.	2368.	1350.	548.	10.10	Packwood Creek @ SCVWD #57
ROUTED TO	3	3311.	2368.	1348.	548.	10.10	
HYDROGRAPH AT	3	10203.	7776.	4725.	2551.	51.84	
ROUTED TO	3	10203.	7776.	4725.	2551.	51.84	
3 COMBINED	3	15038.	13175.	8047.	4190.	192.72	Inflow to Anderson Reservoir
ROUTED TO	3	2985.	2847.	2390.	2045.	192.72	Outflow from Anderson Reservoir
ROUTED TO	4	2803.	2729.	2358.	2016.	192.72	
HYDROGRAPH AT	4	3572.	2471.	1344.	674.	12.55	
ROUTED TO	4	3572.	2471.	1344.	674.	12.55	
2 COMBINED	4	4110.	3115.	2687.	2193.	205.27	Coyote Creek u/s Fisher Cr.
HYDROGRAPH AT	41	2100.	1227.	770.	313.	6.70	Upper Fisher Creek @ Kalana Ave.
ROUTED TO	42	1603.	1224.	763.	312.	6.70	
HYDROGRAPH AT	42	2267.	1402.	820.	323.	6.68	Middle Fisher Creek @ Bailey St.
ROUTED TO	42	2267.	1402.	820.	323.	6.68	
2 COMBINED	42	3846.	2604.	1581.	635.	13.38	
ROUTED TO	4	3406.	2606.	1576.	635.	13.38	
HYDROGRAPH AT	4	934.	546.	294.	119.	2.40	
ROUTED TO	4	934.	546.	294.	119.	2.40	
2 COMBINED	4	3834.	3098.	1861.	753.	15.78	Fisher Creek u/s Coyote Creek
2 COMBINED	4	7943.	6103.	3939.	2878.	221.05	Coyote Creek d/s Fisher Creek
ROUTED TO	5	5330.	5055.	3728.	2840.	221.05	
HYDROGRAPH AT	5	2162.	1283.	748.	361.	8.00	
ROUTED TO	5	2162.	1283.	748.	361.	8.00	
2 COMBINED	5	6432.	6121.	4408.	3145.	229.05	Coyote Creek @ Edenvale (SCVWD #58)
ROUTED TO	6	6099.	5881.	4368.	3138.	229.05	
HYDROGRAPH AT	6	614.	355.	186.	79.	2.18	
ROUTED TO	6	614.	355.	186.	79.	2.18	
HYDROGRAPH AT	6	197.	152.	82.	42.	0.83	
HYDROGRAPH AT	6	120.	92.	48.	22.	0.56	
2 COMBINED	6	312.	239.	130.	65.	1.39	
ROUTED TO	6	151.	151.	129.	65.	1.39	
2 COMBINED	6	762.	501.	315.	144.	3.57	
2 COMBINED	6	6452.	6215.	4615.	3244.	232.62	Coyote Creek u/s Silver Creek
HYDROGRAPH AT	6	1832.	1149.	600.	285.	5.96	
ROUTED TO	16	950.	950.	592.	284.	5.96	Upper Silver Creek u/s Coyote Cr.

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ROUTED TO             7     6887.     6649.     5058.     3475.    238.58   Coyote Creek u/s Silver Creek
HYDROGRAPH AT         7     1067.      820.      444.      228.      4.73
HYDROGRAPH AT         7      638.      491.      254.      120.      3.15
 2 COMBINED           7     1679.     1284.      697.      348.      7.88
ROUTED TO             7      850.      844.      696.      348.      7.88
 2 COMBINED           7     7695.     7478.     5634.     3716.    246.46   Coyote Creek @ 280
ROUTED TO             8     7397.     7214.     5577.     3712.    246.46
HYDROGRAPH AT         8      178.      137.       74.       38.      0.83
HYDROGRAPH AT         8      106.       82.       43.       20.      0.55
 2 COMBINED           8      280.      214.      116.       58.      1.38
ROUTED TO             8      147.      146.      116.       58.      1.38
 2 COMBINED           8     7537.     7344.     5666.     3752.    247.84   Coyote Creek u/s Lower Silver Cr.
HYDROGRAPH AT         8     5477.     5187.     4091.     3304.     43.50   Lower Silver Creek u/s Coyote Cr.
 2 COMBINED           8    12058.    11872.     9642.     6171.    291.34   Coyote Creek d/s Lower Silver Cr.
HYDROGRAPH AT         8     4635.     3878.     2155.     1592.     23.91   Upper Penitencia Creek u/s Coyote Cr.
 2 COMBINED           8    14549.    14277.    11679.     6903.    315.25   Coyote Creek d/s Upper Penitencia Cr.
ROUTED TO             9    14456.    14245.    11634.     6896.    315.25
HYDROGRAPH AT         9      531.      408.      221.      113.      2.44
HYDROGRAPH AT         9      319.      245.      127.       60.      1.63
 2 COMBINED           9      837.      640.      347.      173.      4.07
ROUTED TO             9      436.      433.      346.      173.      4.07
 2 COMBINED           9    14875.    14665.    11930.     7020.    319.32   Coyote Creek @ Montague Expressway
ROUTED TO            10    14831.    14640.    11912.     7017.    319.32
HYDROGRAPH AT        10      289.      222.      120.       62.      1.38
HYDROGRAPH AT        10      173.      133.       69.       33.      0.92
 2 COMBINED          10      455.      348.      189.       94.      2.30
ROUTED TO            10      245.      243.      189.       94.      2.30
 2 COMBINED          10    15066.    14868.    12064.     7084.    321.62   Coyote Creek @ 237
```

LOWER SILVER CREEK HYDROLOGY

Determination of Design Flows

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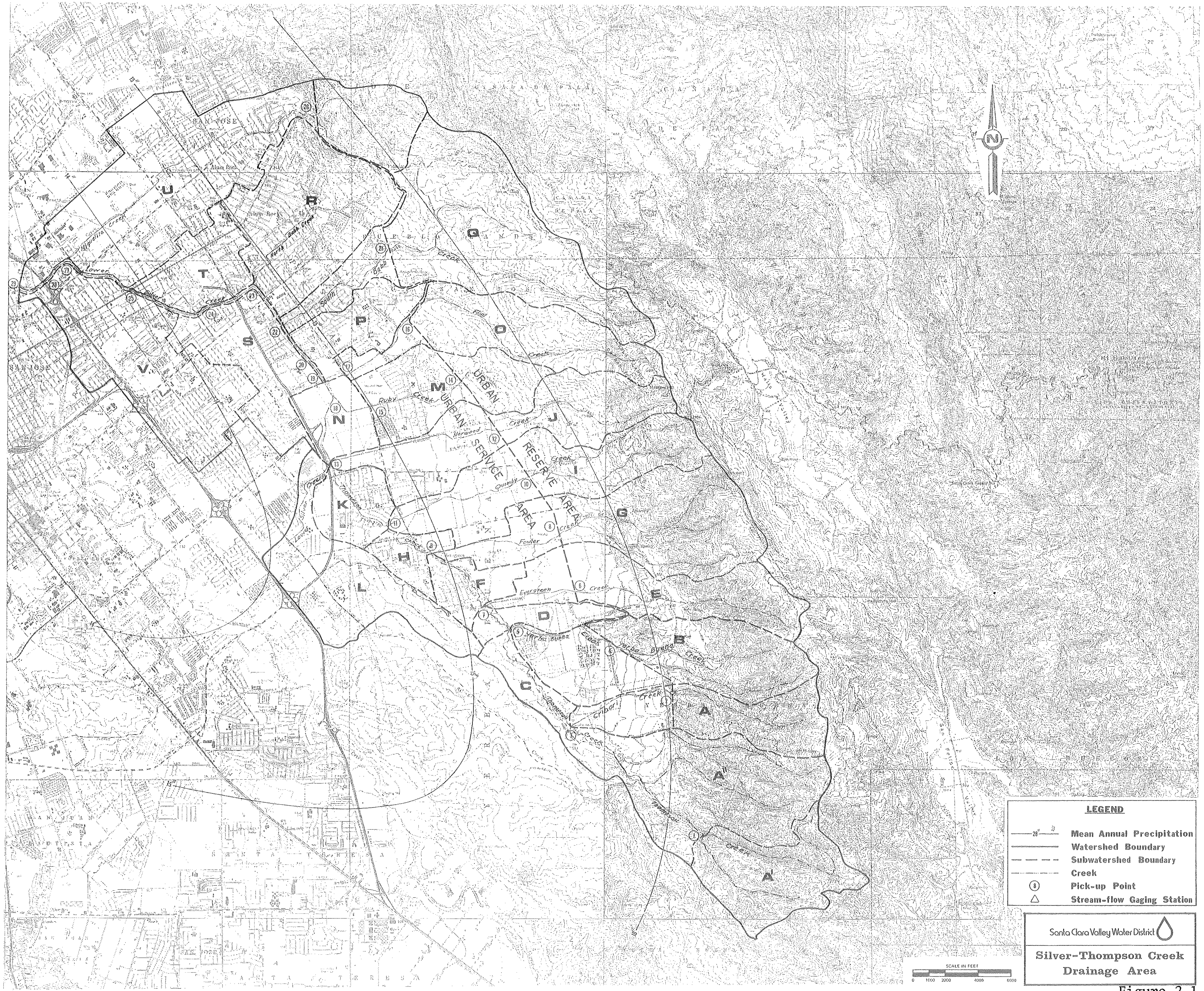
GENERAL INFORMATION

Lower Silver Creek and Thompson Creek are located in the eastern portion of the Santa Clara Valley, partially in the City of San Jose and partially in the unincorporated area of Santa Clara County. Thompson Creek originates in the Diablo Range foothills at an elevation of about 2300 feet and flows northerly through the Evergreen area to its confluence with Lower Silver Creek near the Eastridge Shopping Center. Lower Silver Creek originates near Silver Creek Road at an elevation of about 180 feet and flows northerly through the Lake Cunningham sarea to its confluence with Coyote Creek at an elevation of 64 feet, 1800 feet downstream of U. S. Highway 101. The Lower Silver Creek watershed, shown in Figure 2-1, covers a drainage area of about 44 square miles, including the 17.5 square mile watershed of its major tributary, Thompson Creek. Other tributaries to Lower Silver Creek are from north to south: Miguelita Creek, North Babb Creek, South Babb Creek, Flint Creek, Ruby Creek and Norwood Creek. The locations of these tributaries are listed below:

Tributary Creek	Point of Confluence with Lower Silver Creek
Miguelita Creek	4050 feet upstream of Coyote Creek
North Babb Creek	1200 feet upstream of Story Road
South Babb Creek	2000 feet upstream of Story Road
Flint Creek	500 feet upstream of Cunningham Avenue
Ruby Creek	1100 feet upstream of Cunningham Avenue
Norwood Creek	5400 feet upstream of Cunningham Avenue
Thompson Creek	5500 feet upstream of Cunningham Avenue

The upper region of the watershed is located in steep foothills, while the flatter lower region, about one-third of the watershed, is primarily urbanized. The mean annual precipitation for the watershed varies from 14 inches to 18 inches. Although there are no reservoirs on Lower Silver and Thompson Creeks, some flood retention will be provided by the proposed Lake Cunningham recreation facility at the intersection of Cunningham Avenue and White Road.



Silver-Thompson Creek Drainage Area

Figure 2-1

DISCUSSION OF PROCEDURES

Basin Characteritstics and Preliminary 1% Regional Flows

The Lower Silver Creek watershed was divided into two major watersheds, Lower Silver Creek rural and Lower Silver Creek urban. The rural subareas include A^1, A^{11}, and rural sections of Subareas A, B, E, G, I, J, M, O, Q, and U. Urban subareas include C, D, F, H, K, L, N, P, R, S, T, and V (Figure 2-1). Table 2-1 shows a partial list of typical rural and urban subareas with their respective basin characteristics and the preliminary 1% flows as calculated directly by the regional equations.

As will be shown later, these preliminary 1% values are compared with 1% flows obtained from rainfall by employing the unit hydrograph theory and HEC-1 program. Although this latter technique is used mainly to define design hydrographs, the resultant peak flows are compared and balanced against the regional ones. The end results of the above procedure are further compared with results from the site specific analyses of recorded data if available.

Urban areas are treated based on the District's method of urban hydrology. The density of land use is determined by the ratio of impervious to pervious parts.

Supplementary Analysis

Inherent in the District's hydrology methods are checking procedures that use additional data wherever available. Systematic records, historic data, and stochastic data are utilized.

Because no reliable historic or systematic stream gage data is available for Lower Silver and Thompson Creeks, it was not possible to check the regional regression results with gage analyses. No stochastic data could be generated for the creeks because of this absence of gage data.

Table 2-1

BASIN CHARACTERISTICS

Location	Sub-area	*Drainage Area (sq. mi.)	Mean Annual Precipitation (in.)	Lls (mi.^{2}/ft/ft$^{1/2}$)	Tc (hours)	Q 1% (cfs)
Thompson Crk d/s San Felipe Ck	A^1	1.53	16.9	6.5	.45	579
Thompson Creek U/S Cribari Ck	A^{11}	2.84	16.4	30.7	.67	1047
Cribari Creek U/S Thompson Creek	A	.74	17.0	4.2	.26	328
Yerba Buena Ck U/S Thompson Ck	B	1.59	16.9	6.9	.33	599
Thompson Crk U/S Yerba Buena Creek	C	Ai= .26 Ap= .17	15.3	-	Ti=.74 Tp=.95	-
Thompson Crk U/S Evergreen Creek	D	Ai= .28 Ap= .18	15.5	-	Ti=.69 Tp=.90	-
Evergreen Creek U/S Thompson Ck	E	1.61	16.9	12	.52	642
Thompson Creek U/S Quimby Crk	H	Ai= .27 Ap= .18	14.8	-	Ti=.73 Tp=.94	-
Norwood Creek U/S Thompson Creek	J	1.17	16.9	6.1	.29	479
Flint Creek U/S Silver Creek	O	1.69	16.8	15	.49	680
North Babb Creek U/S Silver Creek	R	Ai=1.54 Ap-1.03	15.3	-	Ti=1.16 Tp=1.36	-
Silver Creek at Alum Rock Avenue	T	Ai= .58 Ap= .53	14.5	-	Ti= .68 Tp= .88	-

*Ai = impervious urban area
Ap = pervious urban area

Las Animas Adjustment Factor

There were, however, extensive gage analyses performed in the adjacent watershed of Coyote Creek, leading to the formulation of the Las Animas adjustment (refer to preceding Coyote Creek Hydrology section).

Because the upper watershed of Lower Silver Creek falls in a meteorologically and geomorphically similar region to Coyote Creek, the Las Animas adjustment was applied to the 1976 regional regression equations for the rural subareas of the Lower Silver Creek Watershed. The following Table 2-2 lists Las Animas adjustments for regional regression constants.

TABLE 2-2

Las Animas Adjustment for
Regional Regression Constants

	Peak	24-Hour Volume	2-Day Volume	3-day Volume
1% Flow	.80	.81	.93	.93
10% Flow	.80	.82	.90	.90

Storm Duration and Distribution

Based on the previously cited Coyote Creek Hydrology section, the storm duration and distribution chosen for the Lower Silver Creek Watershed was the 72 hours Corps of Engineer's Plate 18 derived from the 1958 storm at Hollister, Freedom and Slayton Mine, and presented in Reference 6. This storm as balanced with regional peak, 24-hour, 48-hour, and 72-hour volumes proved to be more critical than the 24-hour storm on Coyote Creek hence Silver Creek.

Five conditions for Lower Silver Creek were analyzed using the 1972 Regional Regression Equations and the 1972 Preliminary Urban Hydrology Methodology. Four of these conditions assumed local urban flooding, and one condition assumed overbanking, i.e., no local urban flooding because streets were assumed to act as flood control facilities and their flows entered creeks by flowing over creek banks. The following is a list of the five conditions studied:

1. As existing urban condition without Lake Cunningham

2. As existing urban condition with Lake Cunningham

3. Complete urbanization of the Urban Service Areas as adopted by San Jose City Council in November 1974 without Lake Cunningham.

4. As above with Lake Cunningham.

5. As above with Lake Cunningham and no local flooding (i.e. overbanking of flood flows).

The documentation of results of the above study is contained in a draft report, "Silver Creek Hydrology, March 1975". Out of these alternatives, Number 4 was used in the already presented 1976 work.

LAKE CUNNINGHAM ANALYSIS

The hydraulic design of the proposed Lake Cunningham Project on Lower Silver Creek was checked by routing the 1% Design Flood Hydrograph inflowing through the project area. The major goal of the investigation was the determination of the peak flow downstream of Cunningham Avenue.

In order to route the inflowing hydrographs of Ruby/Flint Creek and Silver Creek through the channel controls, the weirs, and the lake, storage-discharge curves had to be developed for the proposed lake and stream designs. The storage-discharge curves for the Silver Creek at the control structure and for the Lake Cunningham outlet works were developed by first calculating a storage-elevation and an elevation discharge curve for each structure.

For Silver Creek at the control structure, the storage-elevation curves were calculated first by hand, then by using HEC-2. The calculations were performed by dividing the Silver Creek channel reaches, the prismatic volumes of each was related to the respective water surface elevations at the weir. The storage-elevation calculations are shown on Sheets 1, 2 and 3 of Reference 9.

The development of an elevation-discharge curve for Silver Creek was also performed using HEC-2. Beginning with the water surface elevations downstream of Cunningham Avenue (as computed by Bruce McClish of the Santa Clara Valley Water District), the water surface elevations upstream of the bridge, for various flow rates, were determined (Figure 2-2). The nonlinear elevation-discharge curves are different from Nolte's 1977 design analysis (who assumed a linear relationship).

P-1 SANTA CLARA VALLEY WATER DISTRICT, San Jose, California

By SMT Date 4/78 Chkd. By ________ Date ________ Sheet No. 5 of ____

Subject ________ Job No. ________

CUNNINGHAM AVENUE BRIDGE

ELEVATION Vs DISCHARGE

- ⊙ Present analysis, water surface elevations u/s bridge
- △ Nolte analysis, water surface elevations u/s bridge
- ⊡ McClish backwater analysis, water surface elevations d/s bridge

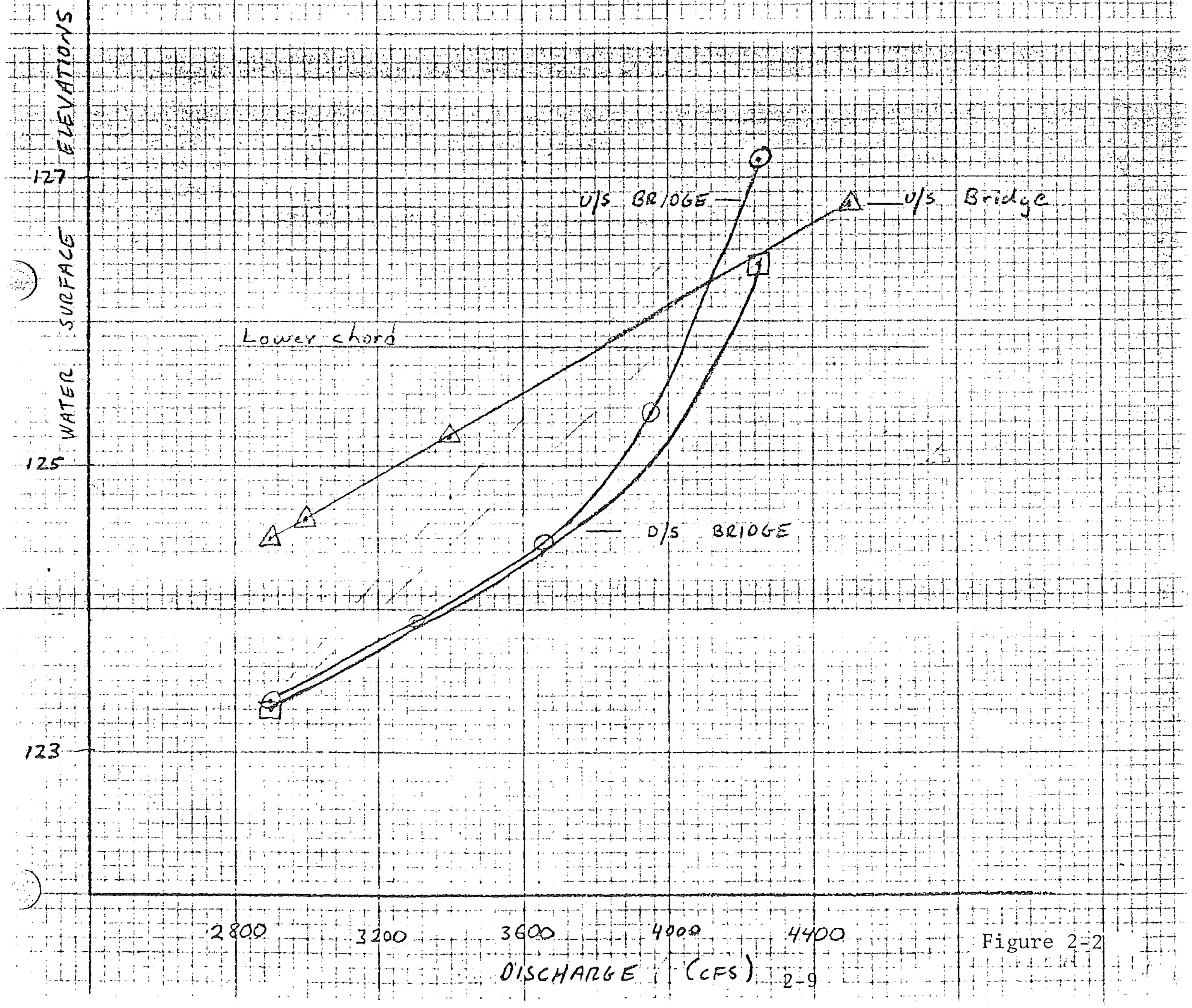


Figure 2-2

The calculation of an elevation-discharge curve for the control structure was begun with these elevation-discharge-relationships for Cunningham Avenue bridge. The results of two different computer runs are shown on Figure 2-3, along with Nolte's results. Curve A was calculated based on Nolte's downstream elevations, which did not accurately reflect the results of the backwater analysis performed on Lower Silver Creek below Cunningham Avenue. Curve B used a W. S. elevation of 123.3 feet upstream of the bridge. Curve B was thought to most closely approximate the actual operation of the control structure and the bridge and was the curve used for later calculations.

Sheet 8 (Reference 9) shows the elevation-discharge curve for the Silver Creek weir as calculated based on a broad-crested, free-flowing weir. The combination of the weir and the control structure elevation-discharge curves is shown on Sheet 9. After relating elevation to storage using the storage-elevation curve, the final storage-discharge relationship for Silver Creek was computed. The result is shown on Figure 2-4. This curve was used with the HEC-1 program to route the inflowing design flow hydrograph through the channel storage.

A similar process was followed on Ruby/Flint Creek, except that it was felt that channel storage was small enough to allow the inflow hydrograph to be routed through the weir and control structure by hand; therefore, no storage-elevation calculations were necessary. The elevation-discharge curve for the Ruby/Flint Creek structures is shown on Sheets 13, 14 and 15 (Reference 9).

The elevation-discharge curve for Ruby/Flint Creek and the storage-discharge curve for Silver Creek were then used to divide the inflowing design flow hydrographs into a weir discharge hydrograph portion and a control structure hydrograph portion. This flow routing was performed by hand and is shown on Sheets 16, 17 and 18 (Reference 9).

P-1 **SANTA CLARA VALLEY WATER DISTRICT, San Jose, California**

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ELEVATION vs. DISCHARGE
LOWER SILVER CREEK CONTROL

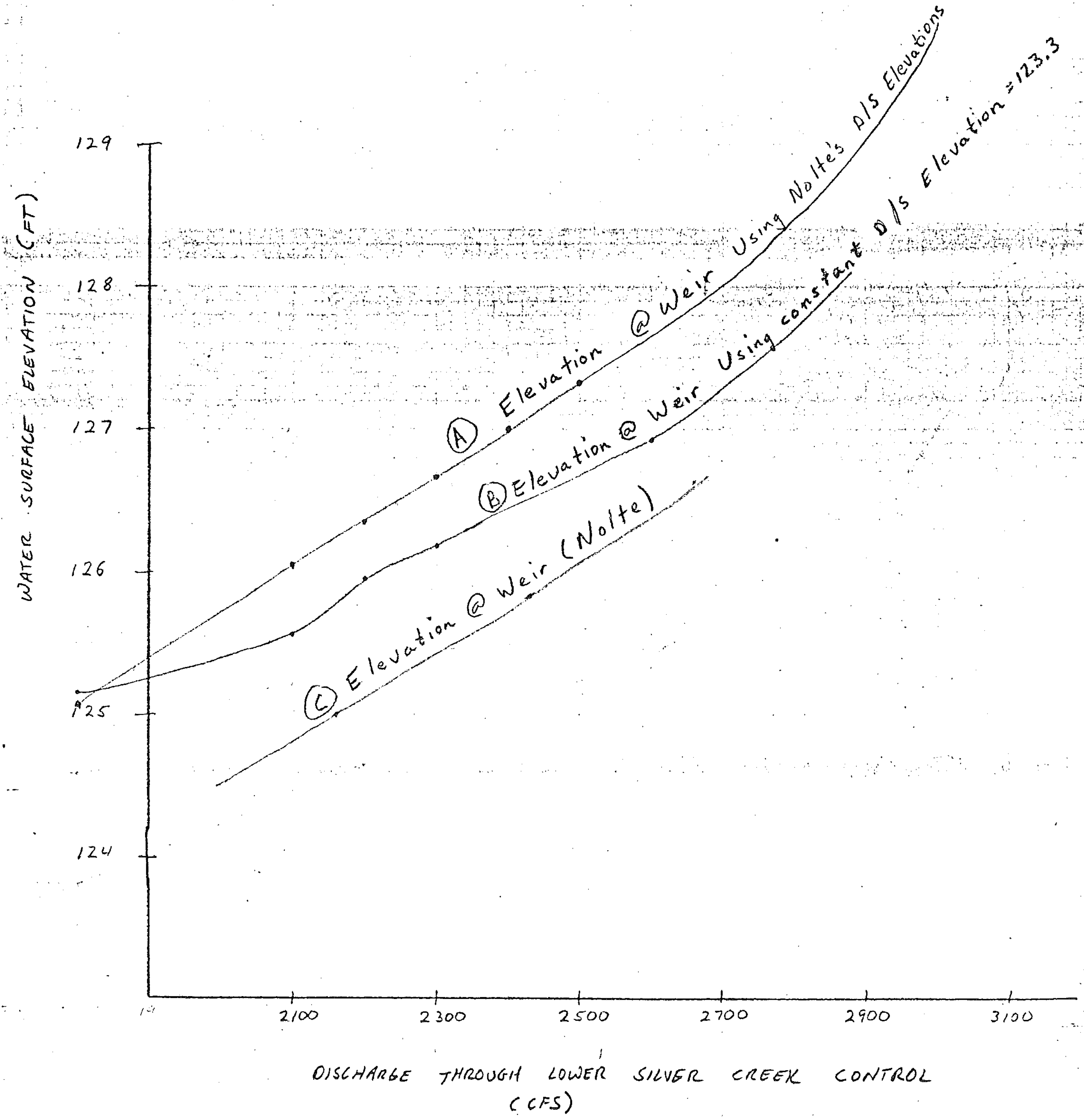

Figure 2-3

SANTA CLARA VALLEY WATER DISTRICT, San Jose, California

By SMT Date 4/78 Chkd. By____ Date____ Sheet No.____ of____

Subject____ Job No.____

CHANNEL STORAGE vs. DISCHARGE

LOWER SILVER CREEK

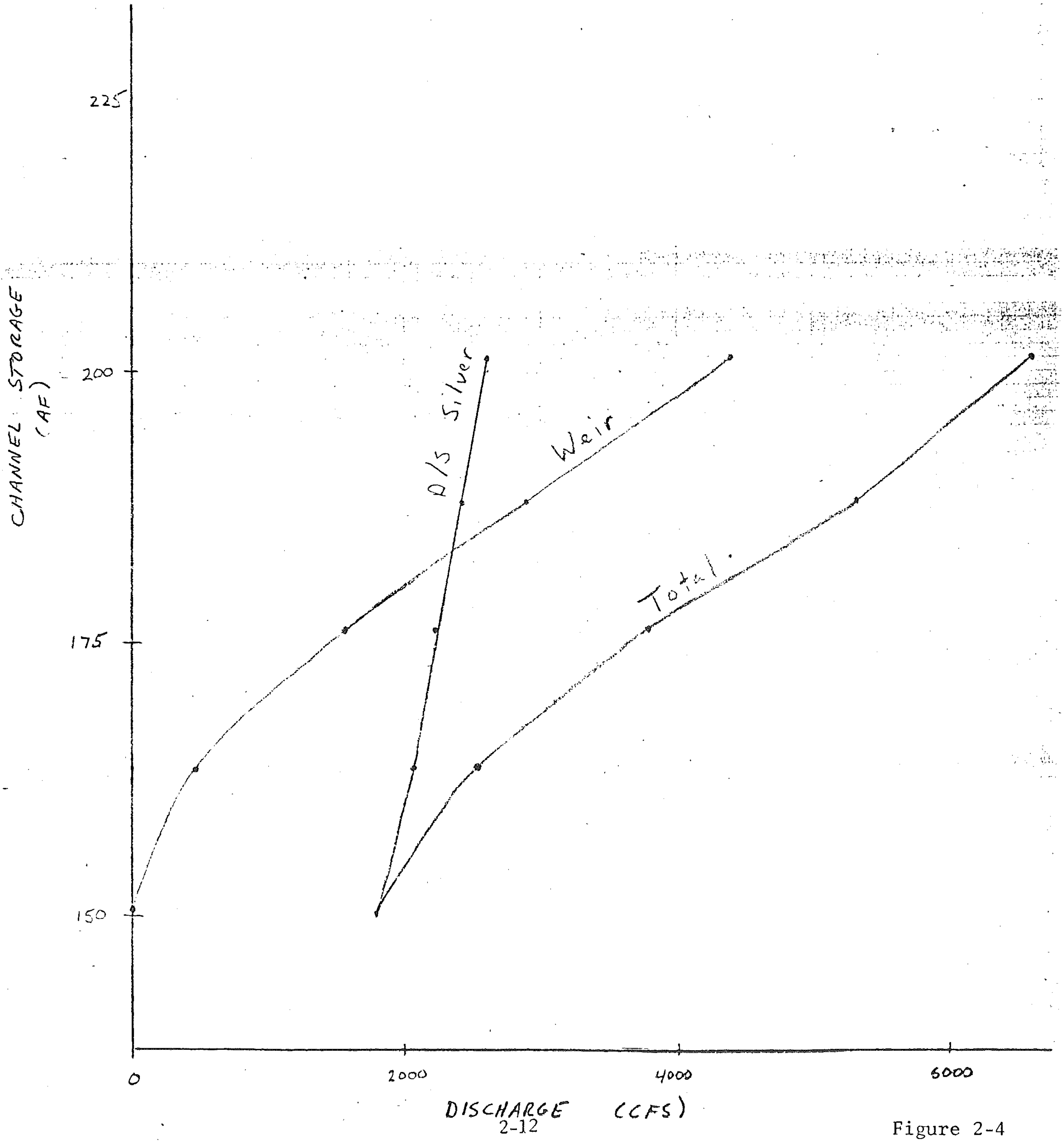

Figure 2-4

Lake Cunningham outlet structure was computed using the storage-elevation curve supplied by Nolte (Sheets 19 and 20) and using the elevation-discharge relationship from Nolte (Sheet 21). This curve was modified to reflect the fact that, above a water surface elevation of 125 feet, overbank flow will occur from the lake into Silver Creek just upstream of Cunningham Avenue. The modification of the curve was sized in such a way as to produce a total flow through Cunningham Avenue bridge which corresponds with the water surface elevation produced by this overbank flow. Thus, the lake's storage-discharge curve is directly related to the combined hydrograph from the Ruby/Flint and the Silver Creek control structures. When the lake floods its bank here, the flow through Cunningham Avenue bridge will be 3,700-3,800 cfs (based on the curve on (Figure 2-1). The flow that does not come from the control structures will come from the lake storage, both over the bank and through the outlet.

The storage-discharge relationship (Figure 2-5) developed from this analysis was used in HEC-1 with the combined Silver Creek and Ruby/Flint Creek weir flow hydrographs and the rainfall over the lake areas as inflows.

The total outflow hydrograph from the Lake Cunningham design was computed by combining the outflow hydrograph from the lake with the hydrographs routed through the Silver Creek and Ruby/Flint Creek control structures. The hydrograph routings are shown on the attached computer printouts and are summarized on Table 2-3.

Summary of Results

1. Inflow to lake will occur when Silver Creek flow exceeds 1,800 cfs or Ruby and Flint Creeks combined flow exceeds 460 cfs.

2. Peak inflows to lake are 1,800 cfs (Silver Creek) and 240 cfs (Ruby/Flint Creeks). Combined peak inflow = 2,000 cfs.

P-1 SANTA CLARA VALLEY WATER DISTRICT, San Jose, California

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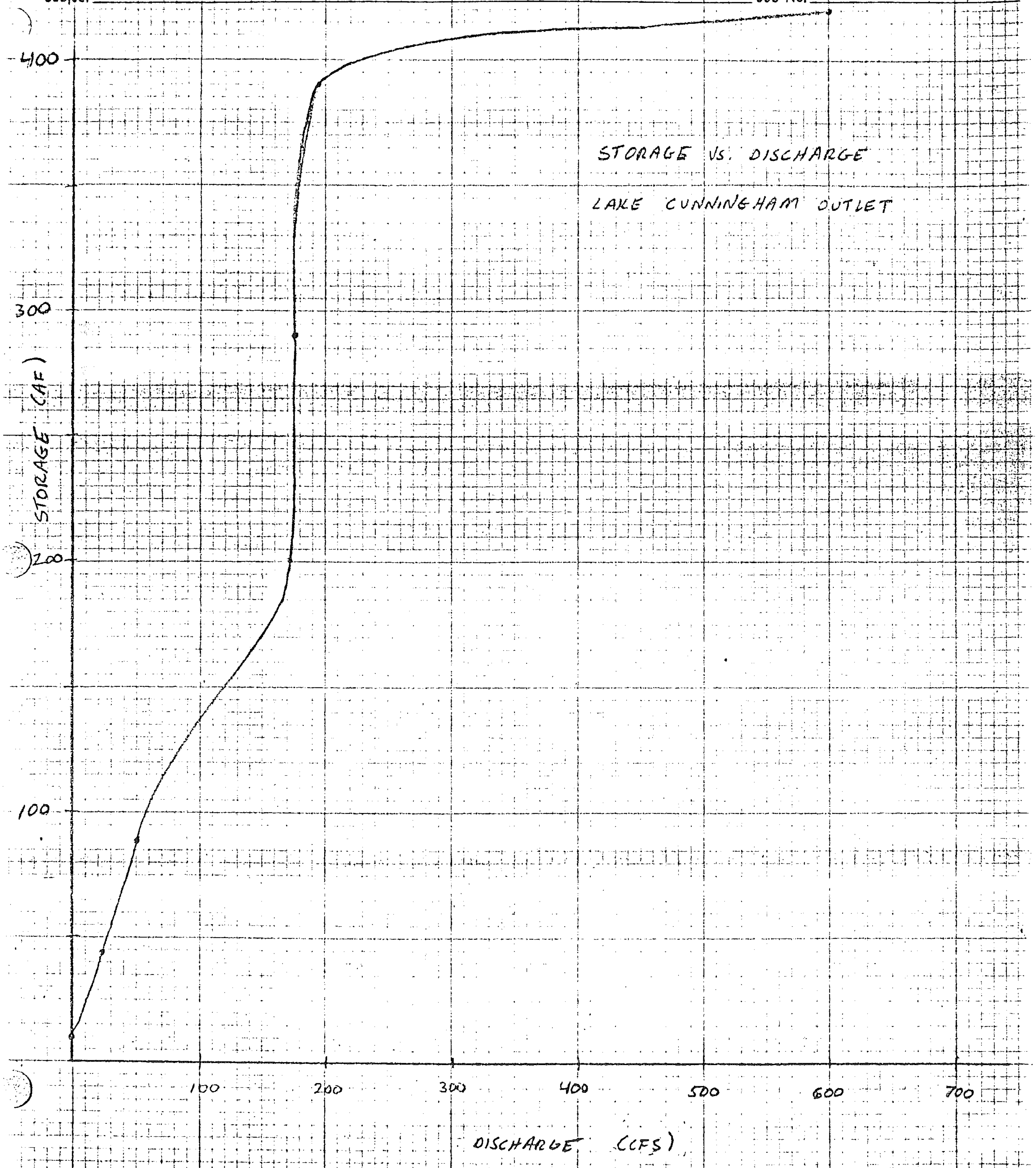

STORAGE VS. DISCHARGE

LAKE CUNNINGHAM OUTLET

Figure 2-5

P-1 SANTA CLARA VALLEY WATER DISTRICT, San Jose, California

By SMV Date 4/78 Chkd. By ____ Date ____ Sheet No. ____ of ____

Subject ____ Job No. ____

HYDROGRAPH ROUTING LAKE ROUTING

Time	Silver Creek			Ruby / Flint Cr.			Lake Cunningham				Silver Cr		
hrs)	Inflow	Elev	Weir	Inflow	Elev	Weir	Total Inflow	Elev	Stor.	Outflow	Elev	Total Outflow	
1	71		0	28		0	0	120	0	0		61	
2	282		0	61		0	0	120	0	0		195	
3	604		0	154		0	0	120	0	0		493	
4	1417	124	0	372	124	0	0	120	0	0		1168	
5	2248	125	0	480	125	15	15	120	0	0		1992	
6	2877	125.5	600	573	125	63	663	120.5	17	5	123	2616	
7	3497	126	1221	709	125.5	174	1395	122	102	67	123	2797	
8	4085	126	1773	732	125.5	192	1965	123.5	235	170	123	2980	
9	3950	126	1734	546	125	42	1776	125	380	197	123	2966	LAKE FULL
10	3176	126	1091	376	725	0	1091	125.5	439	1000	125	3800	
11	2504	125.5	483	339	125	0	483	125.5	422	1000	125	3800	
12	2299	125.5	324	375		0	324	125.5	384	1000	125	3800	
13	2320	125.5	311	369		0	311	125.0	380	365	125	2900	
14	2264	125.5	289	348		0	289	125.0	380	265	124	2802	
15	2131	125.5	226	307			226	125.0	380	360	123	2622	
16	1941	125	116	278			116	125	380	264	123	2431	
17	1800	125	69	263			69	125	378	195	123	2262	
18	1652	125	0	217			0	125	364	192	123	2183	
19	1445	124.0	0	188			0	124.5	349	189		1992	
20	1327			194				124.5	333	186		1832	
21	1300			191				124.5	318	183		1733	
22	1392			261				124	303	180		1783	
23	1678			310				124	288	177		1968	
24	1785			259				124	274	174		2102	
25	1572			188				124	259	173		2036	
6	1282			151				124	245	171		1817	
27	1093			137				124	231	170		1585	
28	1001			134				124	217	168		1421	

Table 2-3

3. Total inflow to lake = 678 acre-feet (Silver) and 42 acre-feet (Ruby/Flint). Peak lake storage = 450 acre-feet.

4. Maximum lake water surface elevation = 125.5 feet.

5. Peak flow downstream Cunningham Avenue = 3,800 cfs. The peak is delayed approximately one hour by the routing.

6. Lake drainage time is approximately 36 hours,

7. The size of the peak flow downstream (3,800 cfs based on this analysis) is very dependent upon the elevation-discharge curve for Cunningham Avenue bridge. Further study in refining this curve could be valuable.

Lake Cunningham is therefore assumed capable of skimming the flood flows about 3,800 cfs and temporarily storing the excess floodwater.

CONCLUSIONS

After careful evaluation of all available data, the design flows recommended in this report are based both on the 1976 regional equations with the Las Animas adjustment and on the 1976 urban hydrology methodology with all urban service areas urbanized. The recommended design storm duration is 72 hours and the recommended storm distribution is the Army Corps of Engineers Plate 18 distribution (Reference 6). With the Lake Cunningham recommended 1% peak outflow of 3,800 cfs, the 1976 recommended 1% and 10% design flows for Lower Silver and Thompson Creeks are listed in Tables 2-4 and 2-5. The values in these tables should be rounded up to significant figures.

Table 2-4

LOWER SILVER CREE
Recommended One Percent Design Flows Complete Urbanization with Lake Cunningham and Los Animas Adjustment

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	463.	292.	138.	103.	1.53	THOMPSON D/S SAN FELIPE CR.
ROUTED TO	3	395.	285.	138.	103.	1.53	
HYDROGRAPH AT	3	838.	543.	264.	197.	2.84	
ROUTED TO	3	838.	543.	264.	197.	2.84	
2 COMBINED	3	1233.	827.	402.	300.	4.37	THOMPSON U/S CRIBARI CR.
HYDROGRAPH AT	2	270.	157.	73.	54.	0.74	
ROUTED TO	3	234.	155.	73.	54.	0.74	
HYDROGRAPH AT	3	113.	78.	36.	27.	0.32	
HYDROGRAPH AT	3	161.	123.	70.	53.	0.47	
2 COMBINED	3	273.	199.	106.	80.	0.79	
ROUTED TO	3	88.	88.	86.	78.	0.79	
2 COMBINED	3	318.	238.	157.	132.	1.53	CRIBARI U/S THOMPSON CR.
2 COMBINED	3	1540.	1064.	557.	432.	5.90	THOMPSON D/S CRIBARI CR.
ROUTED TO	5	1427.	1055.	537.	431.	5.90	
HYDROGRAPH AT	5	61.	41.	19.	14.	0.17	
HYDROGRAPH AT	5	98.	71.	39.	29.	0.26	
2 COMBINED	5	159.	112.	57.	43.	0.43	
ROUTED TO	5	49.	49.	48.	43.	0.43	
2 COMBINED	5	1474.	1102.	604.	474.	6.33	THOMPSON U/S YERBA BUENA CR.
HYDROGRAPH AT	4	479.	296.	143.	107.	1.59	
ROUTED TO	5	421.	294.	143.	107.	1.59	
HYDROGRAPH AT	5	139.	96.	44.	33.	0.40	
HYDROGRAPH AT	5	208.	157.	88.	66.	0.59	
2 COMBINED	5	346.	252.	132.	99.	0.99	
ROUTED TO	5	109.	109.	107.	97.	0.99	
2 COMBINED	5	527.	398.	248.	204.	2.58	YERBA BUENA U/S THOMPSON CR.
2 COMBINED	5	1987.	1499.	852.	678.	8.91	THOMPSON D/S YERBA BUENA CR.
ROUTED TO	7	1974.	1498.	851.	678.	8.91	
HYDROGRAPH AT	7	65.	44.	20.	15.	0.18	
HYDROGRAPH AT	7	105.	77.	42.	31.	0.28	
2 COMBINED	7	170.	120.	62.	46.	0.46	
ROUTED TO	7	52.	51.	51.	46.	0.46	
2 COMBINED	7	2024.	1548.	902.	723.	9.37	THOMPSON U/S EVERGREEN CR.
HYDROGRAPH AT	6	514.	322.	152.	113.	1.61	
ROUTED TO	7	465.	320.	152.	113.	1.61	
HYDROGRAPH AT	7	54.	36.	17.	12.	0.15	
HYDROGRAPH AT	7	85.	62.	33.	25.	0.22	
2 COMBINED	7	139.	98.	50.	37.	0.37	
ROUTED TO	7	39.	39.	39.	36.	0.37	
2 COMBINED	7	504.	358.	190.	149.	1.98	EVERGREEN U/S THOMPSON CR.
2 COMBINED	7	2499.	1901.	1091.	873.	11.35	THOMPSON D/S EVERGREEN CR.
ROUTED TO	9	2468.	1396.	1090.	871.	11.35	
HYDROGRAPH AT	9	76.	52.	24.	18.	0.22	
HYDROGRAPH AT	9	113.	86.	48.	36.	0.33	
2 COMBINED	9	188.	137.	72.	54.	0.55	
ROUTED TO	9	59.	59.	58.	53.	0.55	
2 COMBINED	9	2527.	1954.	1148.	924.	11.90	THOMPSON U/S FOWLER CR.
HYDROGRAPH AT	8	627.	399.	198.	148.	2.26	
ROUTED TO	9	551.	396.	198.	148.	2.26	
HYDROGRAPH AT	9	75.	51.	23.	17.	0.21	
HYDROGRAPH AT	9	120.	87.	47.	35.	0.31	
2 COMBINED	9	195.	138.	70.	52.	0.52	
ROUTED TO	9	40.	40.	40.	37.	0.52	
2 COMBINED	9	590.	435.	237.	185.	2.78	FOWLER U/S THOMPSON CR

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HYDROGRAPH AT      11      62.      42.      19.      14.      0.18
HYDROGRAPH AT      11      92.      70.      39.      29.      0.27
 2 COMBINED        11     154.     111.      58.      43.      0.45
ROUTED TO          11      51.      51.      50.      43.      0.45
 2 COMBINED        11    3133.    2434.    1433.    1151.     19.13 THOMPSON U/S QUIMBY CR.
HYDROGRAPH AT      10     402.     248.     117.      87.      1.31
ROUTED TO          11     341.     249.     117.      87.      1.31
HYDROGRAPH AT      11     137.      81.      37.      28.      0.34
HYDROGRAPH AT      11     189.     141.      77.      58.      0.52
 2 COMBINED        11     305.     222.     114.      85.      0.86
ROUTED TO          11      94.      93.      93.      84.      0.86
 2 COMBINED        11     433.     334.     207.     172.      2.17 QUIMBY U/S THOMPSON CR.
 2 COMBINED        11    3540.    2753.    1640.    1322.     17.30 THOMPSON D/S QUIMBY CR.
ROUTED TO          13    3499.    2755.    1639.    1318.     17.30
HYDROGRAPH AT      13      93.      64.      30.      22.      0.28
HYDROGRAPH AT      13     131.     102.      58.      44.      0.41
 2 COMBINED        13     223.     165.      88.      66.      0.69
ROUTED TO          13      76.      76.      75.      66.      0.69
 2 COMBINED        13    3574.    2829.    1713.    1384.     17.99 THOMPSON U/S NORWOOD CR.
HYDROGRAPH AT      12     383.     239.     110.      82.      1.17
ROUTED TO          13     335.     234.     110.      82.      1.17
HYDROGRAPH AT      13     135.      95.      44.      33.      0.40
HYDROGRAPH AT      13     219.     164.      90.      67.      0.61
 2 COMBINED        13     353.     259.     134.     100.      1.01
ROUTED TO          13     112.     112.     110.      99.      1.01
 2 COMBINED        13     443.     339.     216.     181.      2.18 NORWOOD U/S THOMPSON CR.
HYDROGRAPH AT      13     239.     171.      80.      59.      0.75
HYDROGRAPH AT      13     368.     284.     160.     120.      1.13
 2 COMBINED        13     604.     452.     239.     179.      1.88
ROUTED TO          13     200.     200.     199.     178.      1.88
 3 COMBINED        13    4144.    3347.    2125.    1743.     22.05 THOMPSON D/S NORWOOD CR.
HYDROGRAPH AT      14     190.     105.      50.      37.      0.58
ROUTED TO          15     159.     104.      50.      37.      0.58
HYDROGRAPH AT      15     131.      92.      43.      32.      0.39
HYDROGRAPH AT      15     183.     144.      84.      64.      0.58
 2 COMBINED        15     312.     233.     127.      96.      0.97
ROUTED TO          15     102.     102.     102.      93.      0.97
 2 COMBINED        15     261.     204.     150.     130.      1.55 RUBY @ WHITE RD.
HYDROGRAPH AT      16     544.     344.     162.     120.      1.69
ROUTED TO          17     493.     342.     162.     120.      1.69
HYDROGRAPH AT      17      42.      29.      13.      10.      0.12
HYDROGRAPH AT      17      62.      46.      25.      19.      0.17
 2 COMBINED        17     104.      75.      38.      29.      0.29
ROUTED TO          17      33.      33.      32.      28.      0.29
 2 COMBINED        17     524.     372.     193.     149.      1.98 FLINT @ WHITE RD.
HYDROGRAPH AT      18     231.     174.      98.      74.      0.65 LAKE CUNNINGHAM AREA
 4 COMBINED        18    5056.    4071.    2560.    2096.     26.23 INFLOW TO LAKE AREA.
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Table 2-4 (continued)

Table 2-4 (continued)

LOWER SILVER CREEK
Recommended One Percent
Design Flows Complete Urbanization
With Lake Cunningham and
Los Animas Adjustment

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	18	3800.	3201.	2369.	1921.	26.23	OUTFLOW FROM LAKE CUNNINGHAM
ROUTED TO	20	3800.	3201.	2369.	1921.	26.23	
HYDROGRAPH AT	20	99.	74.	38.	28.	0.35	
HYDROGRAPH AT	20	203.	147.	79.	57.	0.53	
2 COMBINED	20	300.	218.	116.	85.	0.88	
ROUTED TO	20	92.	92.	92.	82.	0.88	
2 COMBINED	20	3892.	3294.	2461.	2003.	27.11	
ROUTED TO	22	3892.	3293.	2460.	2003.	27.11	SILVER U/S SOUTH BABB CR.
HYDROGRAPH AT	21	899.	636.	295.	214.	3.45	
ROUTED TO	22	865.	625.	294.	214.	3.45	
HYDROGRAPH AT	22	77.	53.	24.	18.	0.22	
HYDROGRAPH AT	22	120.	89.	49.	36.	0.33	
2 COMBINED	22	197.	142.	73.	53.	0.55	
ROUTED TO	22	58.	58.	58.	51.	0.55	
2 COMBINED	22	923.	682.	348.	265.	4.00	SOUTH BABB U/S SILVER CR.
2 COMBINED	22	4232.	3873.	2793.	2268.	31.11	SILVER D/S SOUTH BABB CR.
ROUTED TO	23	4215.	3870.	2792.	2267.	31.11	
HYDROGRAPH AT	23	336.	240.	113.	82.	1.03	
HYDROGRAPH AT	23	472.	376.	223.	166.	1.54	
2 COMBINED	23	802.	606.	336.	248.	2.57	
ROUTED TO	23	267.	267.	267.	234.	2.57	NORTH BABB U/S SILVER CR.
2 COMBINED	23	4482.	4137.	3057.	2501.	33.68	SILVER D/S NORTH BABB CR.
ROUTED TO	24	4475.	4136.	3057.	2498.	33.68	
HYDROGRAPH AT	24	199.	137.	64.	46.	0.60	
HYDROGRAPH AT	24	290.	224.	129.	95.	0.91	
2 COMBINED	24	486.	358.	192.	141.	1.51	
ROUTED TO	24	159.	159.	159.	136.	1.51	
2 COMBINED	24	4634.	4294.	3215.	2635.	35.19	SILVER @ JACKSON AVE.
ROUTED TO	25	4620.	4284.	3212.	2625.	35.19	
HYDROGRAPH AT	25	126.	87.	40.	29.	0.38	
HYDROGRAPH AT	25	185.	143.	82.	61.	0.58	
2 COMBINED	25	309.	228.	122.	90.	0.96	
ROUTED TO	25	111.	108.	90.	74.	0.96	
2 COMBINED	25	4722.	4376.	3301.	2700.	36.15	SILVER @ ALUM ROCK AVE.
ROUTED TO	27	4712.	4369.	3299.	2689.	36.15	
HYDROGRAPH AT	26	301.	180.	86.	63.	0.97	
ROUTED TO	27	229.	171.	85.	63.	0.97	
HYDROGRAPH AT	27	435.	316.	150.	109.	1.41	
HYDROGRAPH AT	27	597.	486.	293.	221.	2.11	
2 COMBINED	27	1024.	786.	444.	330.	3.52	
ROUTED TO	27	370.	370.	370.	312.	3.52	
2 COMBINED	27	598.	532.	445.	375.	4.49	MIGUELITA U/S SILVER CR.
2 COMBINED	27	5184.	4891.	3744.	3063.	40.64	SILVER D/S MIGUELITA CR.
ROUTED TO	28	5184.	4891.	3744.	3063.	40.64	
HYDROGRAPH AT	28	350.	250.	116.	84.	1.14	
HYDROGRAPH AT	28	509.	402.	236.	175.	1.72	
2 COMBINED	28	852.	644.	352.	260.	2.86	
ROUTED TO	28	300.	300.	300.	251.	2.86	
2 COMBINED	28	5484.	5191.	4042.	3314.	43.50	SILVER U/S COYOTE CR.
ROUTED TO	29	5477.	5187.	4041.	3304.	43.50	

Table 2-5

LOWER SILVER CREEK
Recommended Ten Percent Design Flows Complete Urbanization With Lake Cunningham and Los Animas Adjustment

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	213.	154.	63.	46.	1.53	THOMPSON D/S SAN FELIPE CR.
ROUTED TO	3	207.	150.	63.	46.	1.53	
HYDROGRAPH AT	3	358.	264.	112.	81.	2.84	
ROUTED TO	3	358.	264.	112.	81.	2.84	
2 COMBINED	3	563.	414.	175.	127.	4.37	THOMPSON U/S CRIBARI CR.
HYDROGRAPH AT	2	126.	84.	34.	24.	0.74	
ROUTED TO	3	122.	83.	34.	24.	0.74	
HYDROGRAPH AT	3	70.	46.	18.	13.	0.32	
HYDROGRAPH AT	3	104.	79.	43.	31.	0.47	
2 COMBINED	3	173.	123.	61.	44.	0.79	
ROUTED TO	3	84.	83.	61.	44.	0.79	
2 COMBINED	3	204.	161.	94.	69.	1.53	CRIBARI U/S THOMPSON CR.
2 COMBINED	3	764.	575.	270.	196.	5.90	THOMPSON D/S CRIBARI CR.
ROUTED TO	5	753.	569.	269.	196.	5.90	
HYDROGRAPH AT	5	38.	24.	9.	7.	0.17	
HYDROGRAPH AT	5	64.	46.	24.	17.	0.26	
2 COMBINED	5	102.	70.	33.	24.	0.43	
ROUTED TO	5	46.	46.	33.	24.	0.43	
2 COMBINED	5	798.	614.	302.	219.	6.33	THOMPSON U/S YERBA BUENA CR.
HYDROGRAPH AT	4	220.	154.	65.	47.	1.59	
ROUTED TO	5	214.	153.	65.	47.	1.59	
HYDROGRAPH AT	5	86.	57.	22.	16.	0.40	
HYDROGRAPH AT	5	135.	101.	54.	39.	0.59	
2 COMBINED	5	220.	157.	76.	55.	0.99	
ROUTED TO	5	105.	104.	76.	55.	0.99	
2 COMBINED	5	319.	253.	140.	102.	2.58	YERBA BUENA U/S THOMPSON CR.
2 COMBINED	5	1115.	866.	442.	322.	8.91	THOMPSON D/S YERBA BUENA CR.
ROUTED TO	7	1111.	865.	442.	322.	8.91	
HYDROGRAPH AT	7	40.	26.	10.	7.	0.18	
HYDROGRAPH AT	7	68.	49.	26.	19.	0.28	
2 COMBINED	7	108.	75.	36.	26.	0.46	
ROUTED TO	7	49.	49.	36.	26.	0.46	
2 COMBINED	7	1160.	912.	478.	348.	9.37	THOMPSON U/S EVERGREEN CR.
HYDROGRAPH AT	6	230.	166.	68.	49.	1.61	
ROUTED TO	7	229.	165.	68.	49.	1.61	
HYDROGRAPH AT	7	33.	22.	8.	6.	0.15	
HYDROGRAPH AT	7	55.	39.	20.	15.	0.22	
2 COMBINED	7	89.	61.	29.	21.	0.37	
ROUTED TO	7	39.	39.	29.	21.	0.37	
2 COMBINED	7	268.	202.	97.	70.	1.98	EVERGREEN U/S THOMPSON CR.
2 COMBINED	7	1426.	1111.	574.	418.	11.35	THOMPSON D/S EVERGREEN CR.
ROUTED TO	9	1417.	1108.	574.	418.	11.35	
HYDROGRAPH AT	9	47.	31.	12.	9.	0.22	
HYDROGRAPH AT	9	73.	55.	29.	21.	0.33	
2 COMBINED	9	120.	85.	41.	30.	0.55	
ROUTED TO	9	58.	58.	41.	30.	0.55	
2 COMBINED	9	1475.	1164.	615.	448.	11.90	THOMPSON U/S FOWLER CR.
HYDROGRAPH AT	8	286.	205.	89.	64.	2.26	
ROUTED TO	9	280.	204.	88.	64.	2.26	
HYDROGRAPH AT	9	46.	30.	12.	8.	0.21	
HYDROGRAPH AT	9	78.	56.	28.	21.	0.31	
2 COMBINED	9	125.	86.	40.	29.	0.52	
ROUTED TO	9	39.	39.	38.	29.	0.52	
2 COMBINED	9	319.	242.	127.	93.	2.78	FOWLER U/S THOMPSON CR.

Table 2-5 (continued)

```
ROUTED TO             11     1782.      1406.      742.       541.      14.68   Thompson O/S Fowler Cr.
HYDROGRAPH AT         11       38.        25.       10.         7.       0.18
HYDROGRAPH AT         11       60.        45.       20.        17.       0.27
 2 COMBINED           11       98.        69.       34.        24.       0.45
ROUTED TO             11       48.        47.       33.        24.       0.45
 2 COMBINED           11     1829.      1450.      775.       565.      15.13   THOMPSON U/S QUIMBY CR.
HYDROGRAPH AT         10      185.       130.       53.        38.       1.31
ROUTED TO             11      179.       128.       53.        38.       1.31
HYDROGRAPH AT         11       72.        48.       19.        14.       0.34
HYDROGRAPH AT         11      123.        90.       47.        34.       0.52
 2 COMBINED           11      195.       138.       66.        48.       0.86
ROUTED TO             11       91.        91.       66.        48.       0.86
 2 COMBINED           11      269.       214.      119.        86.       2.17   QUIMBY U/S THOMPSON CR.
 2 COMBINED           11     2091.      1661.      894.       652.      17.30   THOMPSON O/S QUIMBY CR.
ROUTED TO             13     2071.      1656.      893.       652.      17.30
HYDROGRAPH AT         13       57.        38.       15.        11.       0.28
HYDROGRAPH AT         13       85.        65.       35.        26.       0.41
 2 COMBINED           13      141.       102.       50.        37.       0.69
ROUTED TO             13       72.        72.       50.        37.       0.69
 2 COMBINED           13     2143.      1726.      943.       689.      17.99   THOMPSON U/S NORWOOD CR.
HYDROGRAPH AT         12      177.       127.       50.        36.       1.17
ROUTED TO             13      171.       124.       50.        36.       1.17
HYDROGRAPH AT         13       84.        56.       22.        16.       0.40
HYDROGRAPH AT         13      142.       105.       55.        40.       0.61
 2 COMBINED           13      225.       161.       77.        56.       1.01
ROUTED TO             13      107.       107.       77.        56.       1.01
 2 COMBINED           13      277.       222.      127.        92.       2.18   NORWOOD U/S THOMPSON CR.
HYDROGRAPH AT         13      147.       101.       40.        29.       0.75
HYDROGRAPH AT         13      239.       181.       97.        71.       1.13
 2 COMBINED           13      383.       280.      137.       100.       1.88
ROUTED TO             13      198.       197.      137.       100.       1.88
 3 COMBINED           13     2585.      2131.     1207.       881.      22.05   THOMPSON O/S NORWOOD CR.
HYDROGRAPH AT         14       91.        57.       24.        17.       0.58
ROUTED TO             15       84.        57.       24.        17.       0.58
HYDROGRAPH AT         15       81.        54.       22.        16.       0.39
HYDROGRAPH AT         15      118.        92.       51.        38.       0.58
 2 COMBINED           15      198.       144.       73.        54.       0.97
ROUTED TO             15      102.       102.       73.        54.       0.97
 2 COMBINED           15      183.       153.       97.        71.       1.55   RUBY @ WHITE RD.
HYDROGRAPH AT         16      242.       175.       71.        52.       1.69
ROUTED TO             17      240.       174.       71.        52.       1.69
HYDROGRAPH AT         17       26.        17.        7.         5.       0.12
HYDROGRAPH AT         17       40.        29.       15.        11.       0.17
 2 COMBINED           17       66.        46.       22.        16.       0.29
ROUTED TO             17       31.        31.       22.        16.       0.29
 2 COMBINED           17      270.       203.       93.        67.       1.98   FLINT @ WHITE RD.
HYDROGRAPH AT         18      157.       118.       67.        49.       0.65   LAKE CUNNINGHAM AREA
 4 COMBINED           18     3164.      2587.     1462.      1069.      26.23   INFLOW TO LAKE AREA
```

Table 2-5 (continued)

LOWER SILVER CREEK
Recommended Ten Percent Design Flows Complete Urbanization With Lake Cunningham and Los Animas Adjustment

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	18	3164.	2587.	1462.	1069.	26.23	INFLOW TO LAKE CUNNINGHAM
ROUTED TO	18	2312.	2028.	1435.	1069.	26.23	OUTFLOW FROM LAKE
ROUTED TO	20	2312.	2028.	1435.	1069.	26.23	
HYDROGRAPH AT	20	60.	44.	19.	14.	0.35	
HYDROGRAPH AT	20	129.	90.	44.	32.	0.53	
2 COMBINED	20	187.	132.	63.	46.	0.88	
ROUTED TO	20	92.	92.	60.	44.	0.88	
2 COMBINED	20	2404.	2120.	1495.	1112.	27.11	
ROUTED TO	22	2390.	2116.	1494.	1113.	27.11	SILVER U/S SOUTH BABB CR.
HYDROGRAPH AT	21	393.	319.	130.	94.	3.45	
ROUTED TO	22	389.	313.	130.	94.	3.45	
HYDROGRAPH AT	22	48.	31.	12.	9.	0.22	
HYDROGRAPH AT	22	76.	55.	27.	20.	0.33	
2 COMBINED	22	123.	86.	40.	29.	0.55	
ROUTED TO	22	58.	58.	38.	27.	0.55	
2 COMBINED	22	447.	364.	168.	121.	4.00	SOUTH BABB U/S SILVER CR.
2 COMBINED	22	2554.	2313.	1656.	1234.	31.11	SILVER D/S SOUTH BABB CR.
ROUTED TO	23	2548.	2312.	1656.	1234.	31.11	
HYDROGRAPH AT	23	207.	142.	57.	41.	1.03	
HYDROGRAPH AT	23	293.	228.	125.	93.	1.54	
2 COMBINED	23	497.	364.	183.	134.	2.57	
ROUTED TO	23	267.	266.	174.	127.	2.57	NORTH BABB U/S SILVER CR.
2 COMBINED	23	2815.	2577.	1830.	1360.	33.68	SILVER D/S NORTH BABB CR.
ROUTED TO	24	2809.	2577.	1830.	1360.	33.68	
HYDROGRAPH AT	24	123.	81.	32.	23.	0.60	
HYDROGRAPH AT	24	181.	136.	72.	53.	0.91	
2 COMBINED	24	302.	215.	104.	76.	1.51	
ROUTED TO	24	159.	157.	99.	72.	1.51	
2 COMBINED	24	2968.	2732.	1929.	1432.	35.19	SILVER @ JACKSON AVE.
ROUTED TO	25	2948.	2730.	1927.	1432.	35.19	
HYDROGRAPH AT	25	78.	51.	20.	15.	0.38	
HYDROGRAPH AT	25	115.	87.	46.	34.	0.58	
2 COMBINED	25	192.	137.	66.	48.	0.96	
ROUTED TO	25	88.	56.	50.	42.	0.96	
2 COMBINED	25	2999.	2784.	1977.	1474.	36.15	SILVER @ ALUM ROCK AVE.
ROUTED TO	27	2984.	2782.	1975.	1473.	36.15	
HYDROGRAPH AT	26	140.	95.	39.	28.	0.97	
ROUTED TO	27	123.	90.	39.	28.	0.97	
HYDROGRAPH AT	27	266.	186.	75.	54.	1.41	
HYDROGRAPH AT	27	369.	294.	165.	123.	2.11	
2 COMBINED	27	630.	469.	241.	177.	3.52	
ROUTED TO	27	370.	365.	229.	167.	3.52	
2 COMBINED	27	481.	435.	267.	195.	4.49	MIGUELITA U/S SILVER CR.
2 COMBINED	27.	3402.	3190.	2242.	1669.	40.64	SILVER D/S MIGUELITA CR.
ROUTED TO	28	3402.	3190.	2242.	1669.	40.64	
HYDROGRAPH AT	28	216.	148.	59.	43.	1.14	
HYDROGRAPH AT	28	53.	27.	7.	5.	1.72	
2 COMBINED	28	269.	171.	67.	48.	2.86	
ROUTED TO	28	248.	149.	55.	40.	2.86	
2 COMBINED	28	3471.	3288.	2297.	1708.	43.50	
ROUTED TO	29	3466.	3287.	2296.	1708.	43.50	SILVER @ COYOTE CR.

UPPER PENITENCIA CREEK
HYDROLOGY

Determination of Design Flows

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GENERAL INFORMATION

Upper Penitencia Creek is located in the eastern portion of the Santa Clara Valley, partially in the City of San Jose and partially in the incorporated area of Santa Clara County. Upper Penitencia Creek originates in the Diablo Range foothills at an elevation of about 3000 feet and flows westerly to its confluence with Coyote Creek at an elevation of 75 feet, 3100 feet downstream of King Road. The Upper Penitencia Creek watershed, shown in Figure 3-1, covers a drainage area of about 24 square miles of which 13 square miles are the drainage area of the major tributary , Arroyo Aguague. The majority of the watershed is located in steep foothills, while the flatter flow region, about one-tenth of the watershed, is primarily urbanized. The mean annual precipitation for the watershed varies from 14 inches to 24 inches. Regulation in the watershed is minimal, consisting mainly of Cherry Flat Reservoir with a capacity of 530 acre-feet and a drainage area of 2.2 square miles.

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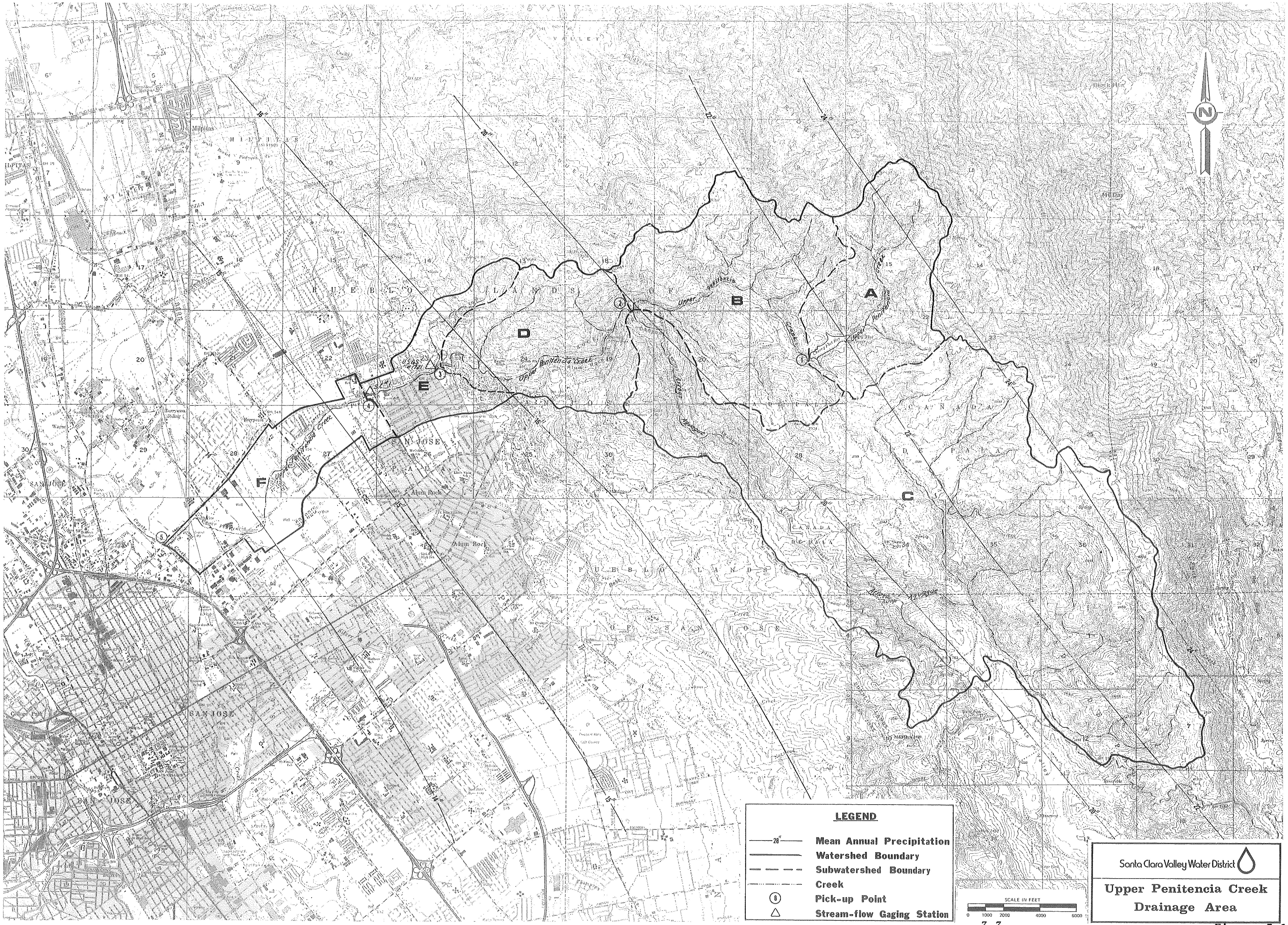

Santa Clara Valley Water District

Upper Penitencia Creek Drainage Area

Figure 3-1

DISCUSSION OF PROCEDURES

Basin Characteristics and 1% Regional Flow

Upper Penitencia Creek Watershed is divided into two major watersheds: Upper Penitencia rural, which includes subarea A; and Upper Penitencia urban, which includes subareas B and C (Figure 3-1). The subareas have the following basin characteristics:

Location	Sub-Area	*Drainage Area (sq. mi.)	Mean Annual Precipitation (in.)	Tc (hours)	Q 1% (cfs)
Upper Penitencia at USGS 1721	A	21.1	21.5	3.0	5870
Upper Penitencia at SCVWD #1	B	Ai=.41 Ap=.68	15.5	Ti = .25 Tp = 1.6	-
Upper Penitencia at Coyote Creek	C	Ai=.81 Ap=.91	14.2	Ti = .25 Tp = 2.5	-

Ai = impervious urban area
Ap = pervious urban area

In rural areas, the preliminary evaluation of design flows from regional regression equations is determined using the drainage areas, mean annual precipitation and basin factors. These preliminary values are compared with peak flows obtained from rainfall by employing the unit hydrograph theory and HEC-1 program. Although this latter technique is used mainly to define design hydrographs, the resultant peak flow is compared and balanced against the regional one. As will be shown later, the end result of the above procedure is further compared with results from the site specific analysis of recorded data.

Urban areas are treated based on the District's method of urban hydrology. The density of land use is determined by the ratio of impervious to pervious parts.

Analysis of Recorded Data and Adjustment of Regional Results

Three stream gaging stations, SCVWD #1, USGS #1721 and SCVWD #2, are located on Upper Penitencia Creek. The first station is located at Piedmont Road, the second at 4000 feet upstream of Piedmont and the third is located upstream of the confluence with Coyote Creek. This last gage was subject to the influence of urbanization and was therefor not used for the statistical analysis.

The maximum annual flow from all of those stations are listed in Table 3-1. The following analyses discuss the various studies done to obtain the information from this data and use it to adjust the information obtainable from the regional equations:

1. Nolte Analysis

In order to expedite the District's hydrology work to match the Federal Insurance Administration study schedule, George S. Nolte and Associates determined the 1976 design flows on Upper Penitencia Creek using the District's 1976 Regional Regression Equations (Reference 1) and Urban Hydrology Methodology (Reference 2). In their hydrology report for Upper Penitencia Creek of September 1977 (Reference 8), they outlined a problem encountered in the watershed when comparing the 1% flow predicted from the 1976 regional regression to the 1% flow predicted from Penitencia gage records. Specifically, at the same location, the regional regression equations were predicting values approximately twice those predicted from gages SCVWD #1 and USGS #1721.

Using weighted gage-regional regression values, Nolte adopted a 1% flow value for their flood insurance study of 3600 cfs at the USGS gage. This value is based on data through 1976 only.

In order to supplement his conclusion, Nolte applied the Soil Conservation Service hydrology method to Upper Penitencia Creek. Using AMC II and a curve numbered 61.3, the Soil Conservation Service value was found equal to their 3600 cfs design flow.

TABLE 3-1
Annual Peak Flows
on
Upper Penitencia Creek

Water Year	USGS (21.1 sq. mi.)	SCVWD No. 1 (22.2 sq. mi.)	SCVWD No. 2 (23.9 sq. mi.)
1943		300	340
44		290	350
1945		350	345
46		105	146
47		0	0
48		3	23
49		51	100
1950		150	60
51		170	310
52		1400	2320
53		22	45
54		0	10
1955		34	90
56		640	1080
57		8	80
58	2100 *	2200	1060
59		19	188
1960		42	215
61		0	0
62	198	-	260
63	295	-	75
64	86	-	80
1965	800	-	970
66	80	28	150
67	1500	1500	1300
68	298	325	410
69	386	400	550
1970	227	123	277
71	519	148	509
72	12	17	21
73	494	224	403
74	651	928	890
1975	376	450	546
76	28	26	27
77	4	4	
78	376	754	

* Historic information supplied to USGS by SCVWD.

2. Application of the Soil Conservation Service Material

The end result of Nolte's work is documented in their FIA report (Reference 8), which concludes that the District method applies only to certain soil groups. However, this conclusion is based on one anomalous watershed and should not be applied generally without further comparative study.

In addition, Nolte's application of the SCS method is limited to finding a Type II antecedent moisture condition (AMC II), which is based on calibrating the local 1964 and 1967 storms and their related floods. AMC II applies only to normal floods. For extreme events such as the 100-year flood, an AMC III should be used, and the curve number of 61.3, originally derived from AMC II, then becomes 79. The one percent peak flow then changes from 3600 cfs to more than 6000 cfs. This indicates that Nolte's study was inconclusive and underestimating the flooding potential of that creek.

3. Final Choice of Design Flows

Nolte was doing this work to expedite the District's hydrology schedule to match that of FIA. The extreme differences that Nolte found instigated an in-depth analysis by the District of Nolte's procedure. The following is a summary of the findings:

Nolte used the recorded data in Table 3-1 for SCVWD #1 in a manner that could be misleading. They excluded the 1948, 1957 and 1977 values on the basis that they are low flows. As a result of this exclusion, they obtained a 1% statistical value equal to 3560 cfs. The District analyzed the same data and found that the low flows did not qualify as low outliers according to the WRC Bulletin #17 (Reference 4). Hence, all the available data was used including the three lower values. The resultant 1% peak flow was estimated at 4000 cfs. Further analysis of the data, including the 1978 data point, yielded a 1% peak flow equal to 4200 cfs.

Table 3-2 shows the District analysis of SCVWD #1 with data through 1978. Because three zero flows were deleted, a WRC recommended adjustment was made to the gage which is shown on Figure 3-2. This adjustment is essentially a curve-fitting procedure which results in the 1% flow as predicted by SCVWD #1 being 4200 cfs. This value is based on 32 years of data. Table 3-3 shows the analysis of USGS gage #1721 with data through 1978. The 1% flow here is predicted to be 3920 cfs, based on 44 years of statistical and historical records.

The District's 1976 regional 1% prediction of 5870 cfs is based on an equivalent 19 years of streamflow data. Table 3-4 shows the results of weighting each of the predicted peak flows with their respective years of record. Based on log weighting 95 years of combined record, as recommended by WRC, the 1% peak flow at the gage site is 4340 cfs. This value is recommended for design purposes and results in a 26% reduction in the original regional value of 5870 cfs.

As shown in Table 3-4, also, the 10% peak flow is recommended at 1370 cfs, which is a 44% reduction in the original regional value of 2440 cfs.

Based on the same ratio between peak flow and volume reductions as in the Las Animas adjustment (refer to Coyote Creek Hydrology section), the following adjustments were made on Upper Penitencia Creek to the 1976 regional equations for the peaks and volumes at the USGS gage.

Upper Penitencia Creek Adjustment
for Regional Regression Constants

	Peak	24-Hour Volume	2-Day Volume	3-Day Volume
1% Flow	.74	.75	.91	.91
10% Flow	.56	.60	.78	.78

This indicates that Nolte's study underestimates the flooding potentials of this creek.

TABLE 3-2

Flood-Flow Frequency
SCVWD #1
June, 1978

```
                                              N       NQH    NOUTL     IYRA    IPLOT
                                             29         0        0        0        0

   FINAL RESULTS
                                          N=32        3 zero flows deleted

                   DAY       MONTH        YEAR       FLOW      ORDERED         RANK    PLOT POS
                     6           1        1943       300.        2200.            1      0.0333
                     3           2        1944       290.        1500.            2      0.0667
                     2           2        1945       350.        1400.            3      0.1000
                     4           1        1946       105.         928.            4      0.1333
Nolte delete  -     24           3        1948         3.         754.            5      0.1667
                    10           3        1949        51.         640.            6      0.2000
                    17           1        1950       150.         450.            7      0.2333
                    18           1        1951       170.         400.            8      0.2667
                    12           1        1952      1400.         350.            9      0.3000
                    30          12        1952        22.         325.           10      0.3333
                    18           1        1955        34.         300.           11      0.3667
                    23          12        1955       640.         290.           12      0.4000
Nolte delete  -     23           2        1957         8.         224.           13      0.4333
                     2           4        1958      2200.         170.           14      0.4667
                    18           2        1959        19.         150.           15      0.5000
                    10           2        1960        42.         148.           16      0.5333
                    11           7        1966        28.         123.           17      0.5667
                    21           1        1967      1500.         105.           18      0.6000
                    30           1        1968       325.          51.           19      0.6333
                    25           1        1969       400.          42.           20      0.6667
                     1           3        1970       123.          34.           21      0.7000
                     2          12        1970       148.          28.           22      0.7333
                    25          12        1971        17.          26.           23      0.7667
                    27           2        1973       224.          22.           24      0.8000
                     1           4        1974       928.          19.           25      0.8333
                    25           3        1975       450.          17.           26      0.8667
                    13           4        1976        26.           8.           27      0.9000
Not analyzed  -      1          10        1976         4.*          4.           28      0.9333
 by Nolte     -     21           1        1978       754.           3.           29      0.9667

                                                  * does not include estimated
                                                    21 cfs imported water
   MEAN                     2.0746
   STD DEV                  0.7756
   COMPUTED SKEW           -0.3542
   REGIONAL SKEW          -99.0000
   ADOPTED SKEW            -0.3542

                                          n/N = 0.91

   COMPUTED FLOW (cfs)    PROBABILITY                    .05 LIMIT       .95 LIMIT
            9494.              0.002  .0018                 37653.          3734.
            6527.              0.005  .0045                 23566.          2714.
            4732.              0.010  .009                  15783.          2061.
            3285.              0.020  .018                  10038.          1506.
            2154.              0.040  .036                   5967.          1044.
            1082.              0.100  .091                   2581.           568.
             546.              0.200  .181                   1141.           305.
             132.              0.500  .453                    232.            76.
              27.              0.800  .725                     49.            13.
              11.              0.900                           22.             5.
```

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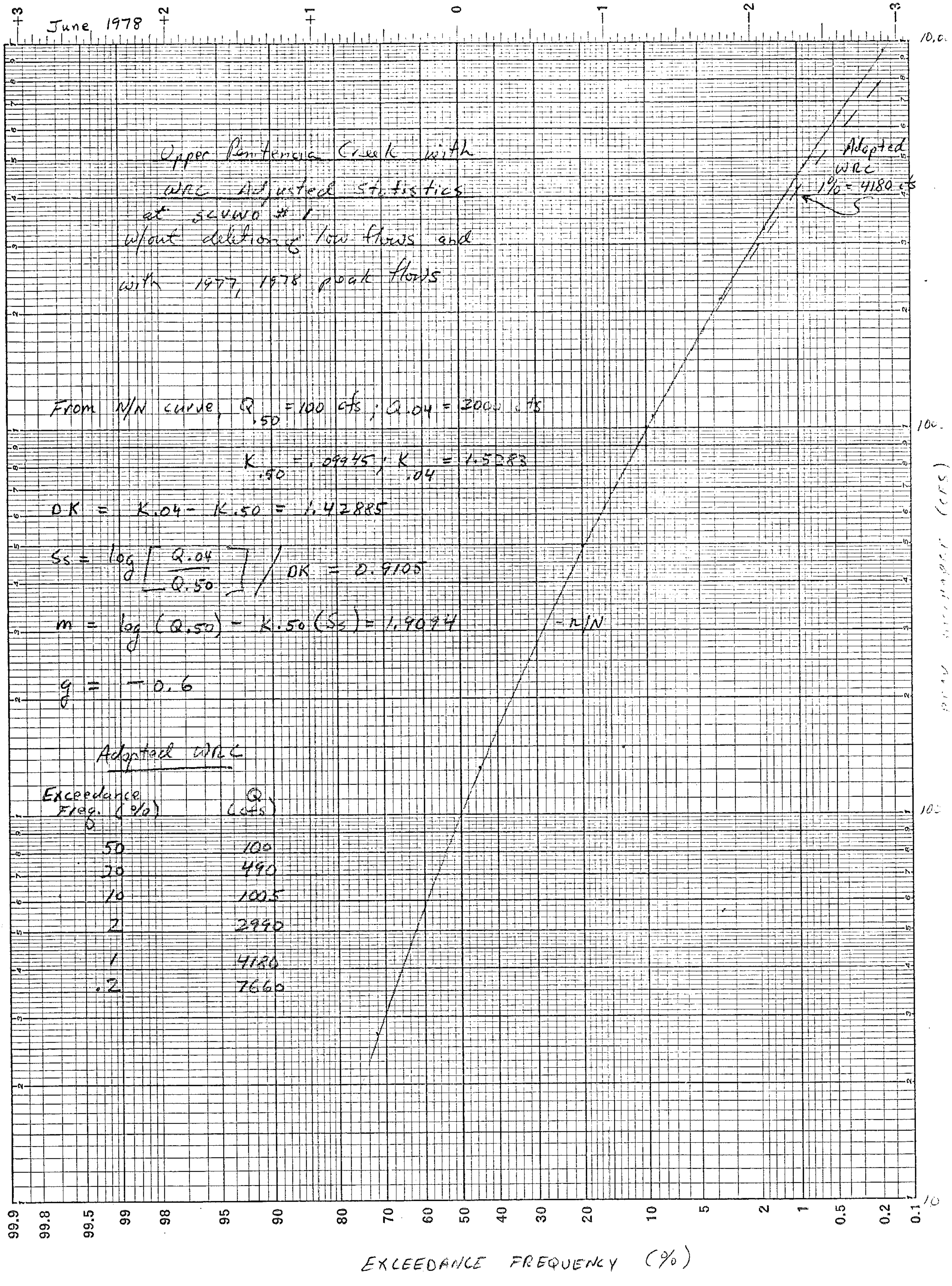

June 1978

Upper Penitencia Creek with
WRC Adjusted Statistics
at SCVWD # 1
w/out deletion of low flows and
with 1977, 1978 peak flows

Adopted
WRC
1% = 4180 cfs

From N/N curve, $Q_{.50} = 100$ cfs ; $Q_{.04} = 2000$ cfs

$K_{.50} = .09945$; $K_{.04} = 1.5283$

$\Delta K = K_{.04} - K_{.50} = 1.42885$

$S_s = \log\left[\frac{Q_{.04}}{Q_{.50}}\right] / \Delta K = 0.9100$

$m = \log(Q_{.50}) - K_{.50}(S_s) = 1.9094$

$g = -0.6$

Adopted WRC

Exceedance Freq. (%)	Q (cfs)
50	100
20	490
10	1005
2	2990
1	4180
.2	7660

Figure 3-2

Table 3-3

Flood-Flow Frequency

USGS # 1721

June, 1978

```
                                              N       NQH    NOUTL      IYRA     IPLOT
                                             17         1        0      1935         0

FINAL RESULTS

                DAY       MONTH       YEAR       FLOW      ORDERED          RANK   PLOT POS
                16            2       1962       198.       2100.              1     0.0222
                28            3       1963       295.       1500.              2     0.0759
                21            1       1964        86.        800.              3     0.1302
                23           12       1964       800.        651.              4     0.1846
                28           12       1965        80.        519.              5     0.2389
                21            1       1967      1500.        494.              6     0.2933
                30            1       1968       298.        386.              7     0.3477
                25            1       1969       386.        376.              8     0.4020
                 1            3       1970       227.        376.              9     0.4564
                 2           12       1970       519.        298.             10     0.5107
                25           12       1971        12.        295.             11     0.5651
                18            1       1973       494.        227.             12     0.6195
                 1            4       1974       651.        198.             13     0.6738
                21            3       1975       376.         86.             14     0.7282
                 2            3       1976        28.         80.             15     0.7826
Not analyzed —   2            1       1977         4.         28.             16     0.8369
by Nolte     —   5            3       1978       376.         12.             17     0.8913
                 2            4       1958      2100.          4.             18     0.9456

MEAN               2.2946
STD DEV            0.6907
COMPUTED SKEW     -1.1837
REGIONAL SKEW     -0.6000
ADOPTED SKEW      -0.6000

COMPUTED FLOW (CFS)   PROBABILITY                    .05 LIMIT          .95 LIMIT
      6204.              0.002                         28779.             2427.
      4869.              0.005                         20770.             1986.
      3921.              0.010                         15539.             1658.
      3040.              0.020                         11069.             1339.
      2240.              0.040                          7387.             1034.
      1329.              0.100                          3737.              658.
       770.              0.200                          1864.              403.
       231.              0.500                           446.              123.
        55.              0.800                           105.               23.
        24.              0.900                            49.                8.
        11.              0.950                            26.                3.
         2.              0.990                             7.                0.
```

TABLE 3-4

Upper Penitencia Peak Flow Analysis with Data through 1978 (cfs)

		USGS #1721		SCVWD #1		Weighted Average		Log Weighted Average		Regional Adjustment
Exceedance Frequency (%)	Regional (19 Yrs)	without 1978 (43 Yrs)	with 1978 (44 Yrs)	without 1978 (31 Yrs)	with 1978 (32 Yrs)	without 1978 93 Yrs	with 1978 95 Yrs	without 1978 93 Yrs	with 1978 95 Yrs	(% Reduced for 1978)
0.2	8520	6510	6200	7210	7660	7150	7160	7120	7100	17
1.0	5870	4065	3920	3995	4180	4410	4400	4350	4340	26
2.0	4770	3130	3040	2835	2990	3370	3370	3300	3310	31
10.0	2440	1340	1330	960	1005	1440	1440	1360	1370	44

For the 1%, it may be seen that the weighted average without 1978 data is almost equal to the log weighted average with 1978 data. This value is 4340 cfs, the District adopted value.

CONCLUSIONS

Using the recommended adjustments to 1% and 10% peak flows and volumes at USGS Gage #1721, downstream flow rates were computed using the SCVWD Urban Hydrology Methodology, and the U. S. Army Corps of Engineers Plate 18 storm distribution (Reference 6). The recommended 1% design flows are summarized in Table 3-5 along with the recommended 10% design flows in Table 3-6. The values in these tables should be rounded up to significant figures.

TABLE 3-5

UPPER PENITENCIA CREEK

RECOMMENDED 1% DESIGN FLOWS

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	4342.	3497.	1822.	1343.	21.10	UPPER PENITENCIA AT USGS #1721
ROUTED TO	1	4342.	3497.	1822.	1343.	21.10	
ROUTED TO	2	4313.	3494.	1822.	1343.	21.10	
HYDROGRAPH AT	2	228.	163.	77.	56.	0.68	
HYDROGRAPH AT	2	127.	101.	60.	45.	0.41	
2 COMBINED	2	355.	263.	137.	101.	1.09	
ROUTED TO	3	169.	168.	137.	101.	1.09	
2 COMBINED	2	4481.	3661.	1957.	1444.	22.19	UPPER PENITENCIA AT GRIDLEY ST.
ROUTED TO	3	4366.	3611.	1952.	1443.	22.19	(AT SCVWD #1)
HYDROGRAPH AT	3	273.	201.	95.	69.	0.91	
HYDROGRAPH AT	3	237.	188.	112.	83.	0.81	
2 COMBINED	3	510.	389.	207.	152.	1.72	
ROUTED TO	3	269.	267.	205.	152.	1.72	
2 COMBINED	3	4635.	3878.	2155.	1596.	23.91	UPPER PENITENCIA AT COYOTE CREEK

TABLE 3-6
UPPER PENITENCIA CREEK
RECOMMENDED 10% DESIGN FLOWS

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	1342.	1178.	563.	438.	21.10	UPPER PENITENCIA AT USGS #1721
ROUTED TO	1	1342.	1178.	563.	438.	21.10	
ROUTED TO	2	1332.	1177.	563.	438.	21.10	
HYDROGRAPH AT	2	139.	95.	38.	29.	0.68	
HYDROGRAPH AT	2	84.	66.	39.	31.	0.41	
2 COMBINED	2	223.	161.	77.	60.	1.09	
ROUTED TO	3 ?	159.	151.	77.	60.	1.09	
2 COMBINED	2	1490.	1325.	639.	498.	22.19	UPPER PENITENCIA AT GRIDLEY ST. (AT SCVWD #1)
ROUTED TO	3	1470.	1302.	637.	497.	22.19	
HYDROGRAPH AT	3	168.	119.	48.	37.	0.91	
HYDROGRAPH AT	3	157.	124.	72.	57.	0.81	
2 COMBINED	3	325.	243.	121.	94.	1.72	
ROUTED TO	3	255.	228.	120.	94.	1.72	
2 COMBINED	3	1717.	1524.	756.	591.	23.91	UPPER PENITENCIA AT COYOTE CREEK

LOWER PENITENCIA AND BERRYESSA CREEKS HYDROLOGY

Determination of Design Flows

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GENERAL INFORMATION

Lower Penitencia Creek has a total drainage area of 28.3 square miles, of which 22 square miles is the drainage area for Berryessa Creek (Figure 4-1). Berryessa Creek Watershed originates at 2,000 feet above mean sea level in the San Francisco East Bay Hills south of Calaveras Reservoir. The drainage course flows northwest through the City of Milpitas. Many tributaries and storm systems drain the urban area of the City and contribute to the Berryessa Creek channel flow. Mean annual precipitation across the Berryessa Creek Watershed ranges from 15 to 22 inches while that of the Lower Penitencia Creek Watershed above Berryessa Creek confluence ranges from 13 to 15 inches. There are no reservoirs in the watersheds. There are storage ponds along Berryessa Creek which have neglible effect on the channel flood flow values.

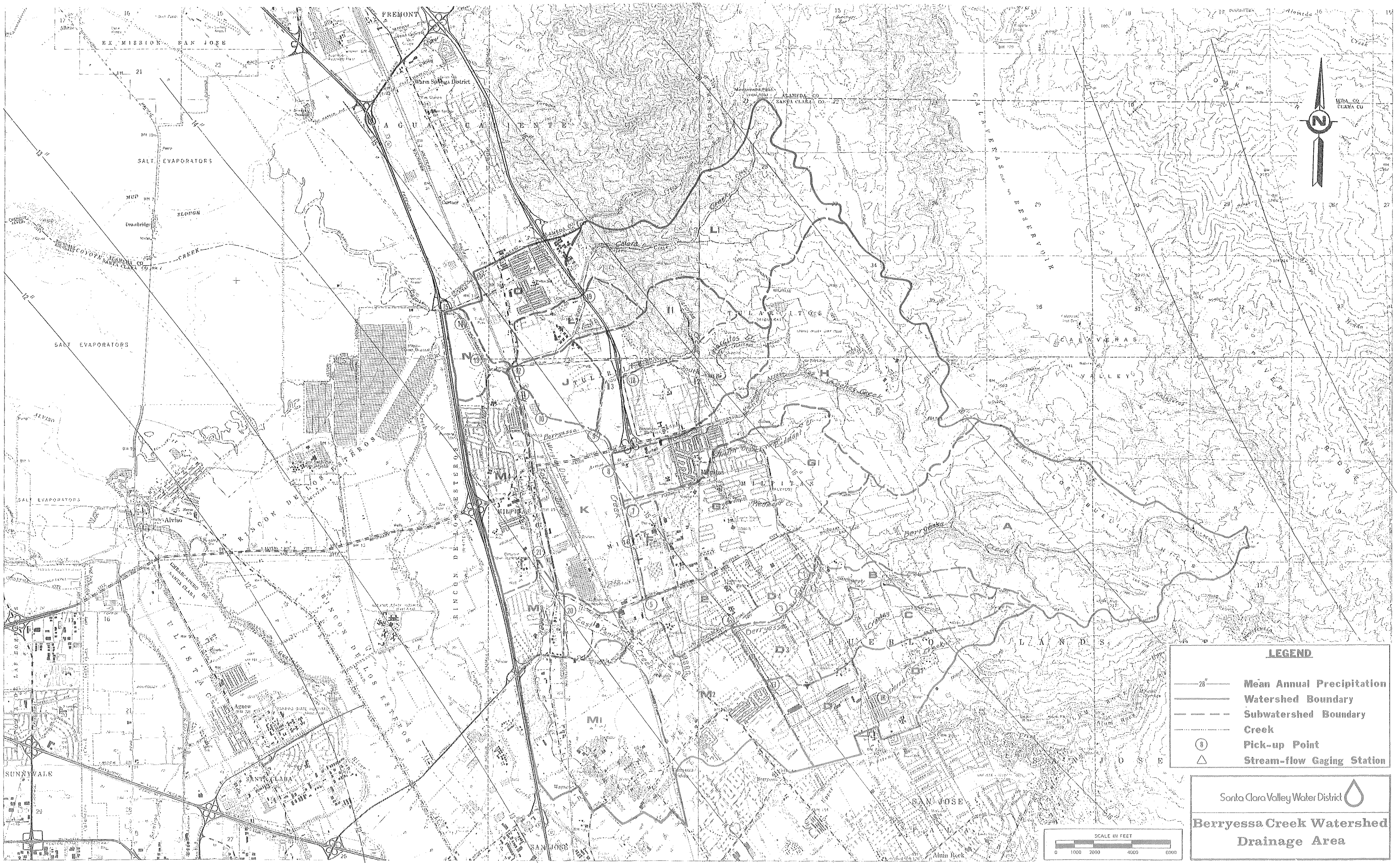
LEGEND
28" Mean Annual Precipitation
Watershed Boundary
Subwatershed Boundary
Creek
8 Pick-up Point
△ Stream-flow Gaging Station
Santa Clara Valley Water District
Berryessa Creek Watershed
Drainage Area
SCALE IN FEET
0 1000 2000 4000 6000
N
FREMONT
AGUA CALIENTE
SALT EVAPORATORS
MUD SLOUGH
COYOTE CREEK
ALVISO
Alviso
RINCON DE LOS ESTEROS
MILPITAS
Milpitas
TULARCITOS
CALAVERAS RESERVOIR
CALAVERAS VALLEY
PUEBLO LANDS
SAN JOSE
SANTA CLARA
SUNNYVALE
Agnew
Berryessa Creek
Calera Creek
Piedmont Creek
Arroyo de los Coches Creek
East Penitencia Creek
Sierra Creek
Crosley Creek
Tularcitos Cr.
Alum Rock
A
B
C
D
E
F
G
H
I
J
K
L
M
N
12"
13"
14"
20"
Ex Mission San Jose

Figure 4-1

DISCUSSION OF PROCEDURES

Basin Characteristics

Berryessa Creek watershed is divided into two major watersheds: Berryessa rural, which includes subareas A, B, C, D, G, H, I1 and L1; and Berryessa urban, which included subareas D2, D3, D4, E, F, G2, I2, I3, J, K, L2, M1, M2, M3, M4, N and O (Figure 4-1). A partial list of the rural and urban subareas has the following basin characteristics.

Location	Sub-area	*Drainage Area (sq. mi.)	Mean Annual Precipitation (in.)	Lls ($mi.^2/ft/ft^{1/2}$)	Tc (hours)
Berryessa Crk @ Piedmont Rd	A	4.51	19.8	56.5	1.11
Sweigert Creek Upstream Berryessa Crk	B	.49	15.9	3.0	.18
Crosley Creek Upstream Berryessa Crk	C	.93	15.8	16.7	.18
Los Coches Crk Upstream Berryessa Crk	H	4.02	18.0	47.5	.98
Calera Creek @ Highway 680	L1	2.43	19.0	14.2	.48
Berryessa Crk @ Landess Ave.	E	Ai =.54 Ap =.23	15.1	-	Ti = .28 Tp = 1.46
Piedmont Crk Upstream Berryessa Crk	G2	Ai =.71 Ap =.34	15.5	-	Ti = .28 Tp = 1.46
Ford & Wrigley Ditches U/S Berryessa Crk	K	Ai =1.11 Ap = .43	14.5	-	Ti = .34 Tp = 3.10
Lower Penitencia Crk U/S Berryessa Crk	M4	Ai =.63 Ap =.54	14.0	-	Ti = .34 Tp = 3.80

*Ai = Impervious Urban Area; Ap - Pervious Urban Area

In rural areas, the preliminary evaluation of design flows from regional regression equations is determined using the drainage areas, mean annual precipitation and basin factors. These preliminary values are compared with peak flows obtained from rainfall by employing the unit/hydrograph theory and HEC-1 program. Although this latter technique is used mainly to define design hydrographs, the resultant peak flow is compared and balanced against the regional one. As will be shown later, the end result of the above procedure is further compared with results from the site-specific analysis of recorded data if available.

Urban areas, which constitute the larger part of the watershed, are treated based on the District's method of urban hydrology (Reference 2). The density of land use is determined by the ratio of impervious to pervious parts.

Recorded Data and Application of Regional Regression Equations

Inherent in the District hydrology method are checking procedures that utilize additional data wherever available. Recorded data, stochastically generated data, and data obtained from rainfall are utilized. Following is a discussion of all these procedures and their application to Berryessa Creek.

There is one stream gage on Berryessa Creek, SCVWD #64, Berryessa Creek upstream of Calaveras Boulevard. This station was installed in 1969 and has seven years of flow data. Due to this short record and because of extensive urbanization upstream of the gage, the annual peak record was found to be too biased for statistical analysis especially in the calculation of the standard deviation. Analysis of the mean recorded data, which is less sensitive to the short (seven-year) record, was compared to the mean predicted by the regional regression equations. These values, shown below, indicate that the prediction by the 1976 regression is acceptable at the stream gage location.

Mean of Log Q's/Q (cfs)	1976 Predicted Mean for Log Q's/Q (cfs)
2.32/210	2.41/255

Using these regression equations the 1% flows @ various rural locations are shown below:

Location	1% Flow from Regression - c.f.s.
Berryessa @ Piedmont	1580
Sweigart @ Berryessa	242
Crosley @ Berryessa	450
Los Coches @ Berryessa	1420
Calera @ Highway 680	892

Flood Estimates from Precipitation

In Santa Clara County the mean annual precipitation (M.A.P.) reflects well the integrated effects of availability of atmospheric moisture, of elevation and topography, and of efficiency of the orographic mechanism, which are significant parameters of storm rainfall. It was, therefore, possible to develop a relationship of storm values vs. M.A.P. for different durations and frequency using data collected from rainfall stations in and around the County. The 24-hour rain with an exceedance frequency of 1% is used as the design storm.

The unit hydrograph theory was employed to convert rainfall to runoff. Clark's synthetic unit hydrograph was used with the Corps of Engineers' 24-hour standard storm distribution and a 30% loss rate. Using this approach, the 1% peak flow rate at the confluence with Coyote Creek is 7,000 cfs. This is a rough figure and a parameter optimization technique would be necessary to refine it further; however, it does provide a guide to prevent gross errors in the final choice of design values.

OTHER STUDIES - PAST AND PRESENT

The hydrology of Berryessa and Lower Penitencia Creeks was studied by the Santa Clara Valley Water District in 1967. This present work, which depends on the 1976 equation, was done by Nolte Engineers as a part of their FIA work. Comparison between the 1967 and 1977 design flow values (Table 4-1) shows that the 1977 values are increased at the rural areas. The increase is reflected throughout the downstream urban watershed.

To check the reason of this increase, the U. S. Geological Survey Regression Equations for the San Francisco Bay Region were applied to rural areas. The resultant 1% flow rate was comparable with the Districts.

TABLE 4-1

Berryessa Creek
Comparative 1% Design Flows

Location	Drainage Area $(mi.^2)$	1967 SCVWD Regression (cfs)	1976 SCVWD Regression (cfs)
Berryessa Creek at Piedmont Road	4.51	1150	1580
Berryessa Creek Downstream Sweigert Creek	5.00	1400	1780
Berryessa Creek Downstream Sierra Creek	7.70	1900	2390
Berryessa Creek @ Landess Ave.	8.76	2000	2550
Berryessa Creek Downstream Piedmont Creek	11.05	2500	2980
Berryessa Creek Downstream Los Coches Creek	15.07	3500	4020
Berryessa Creek Downstream Tularcitos Creek	17.03	4100	4450
Calera Creek at I-680	2.43	700	890
Calera Creek Upstream Berryessa Creek	2.93	750	920
Los Coches Creek Upstream Berryessa Creek	4.02	970	1420
Sierra Creek at Piedmont Road	.82	140	190
Tularcitos Creek Upstream Berryessa Creek	1.98	700	450

CONCLUSIONS

This report presents a review of the information available for analysis of the hydrology of Berryessa and Lower Penitencia Creek Watersheds.

The flow rates have been computed from the 1976 Regional Regression Equations coupled with the urban hydrology methodology and compared with other studies.

The 1% design flow at the Coyote Creek confluence (6980 cfs) obtained from analyses based on the regional regression compares favorably with the value obtained from data generated by rainfall (7000 cfs). The results obtained from the 1976 regional equations and the application of the urban model have therefore been chosen and used here for the sake of homogeneity with other non-instrumented watersheds. Figure 4-2 shows frequency curves at various locations while the 1% and 10% design flows for Berryessa and Lower Penitencia Creeks are summarized in Tables 4-2 and 4-3, respectively. The values in these tables should be rounded up to significant figures.

May 1978

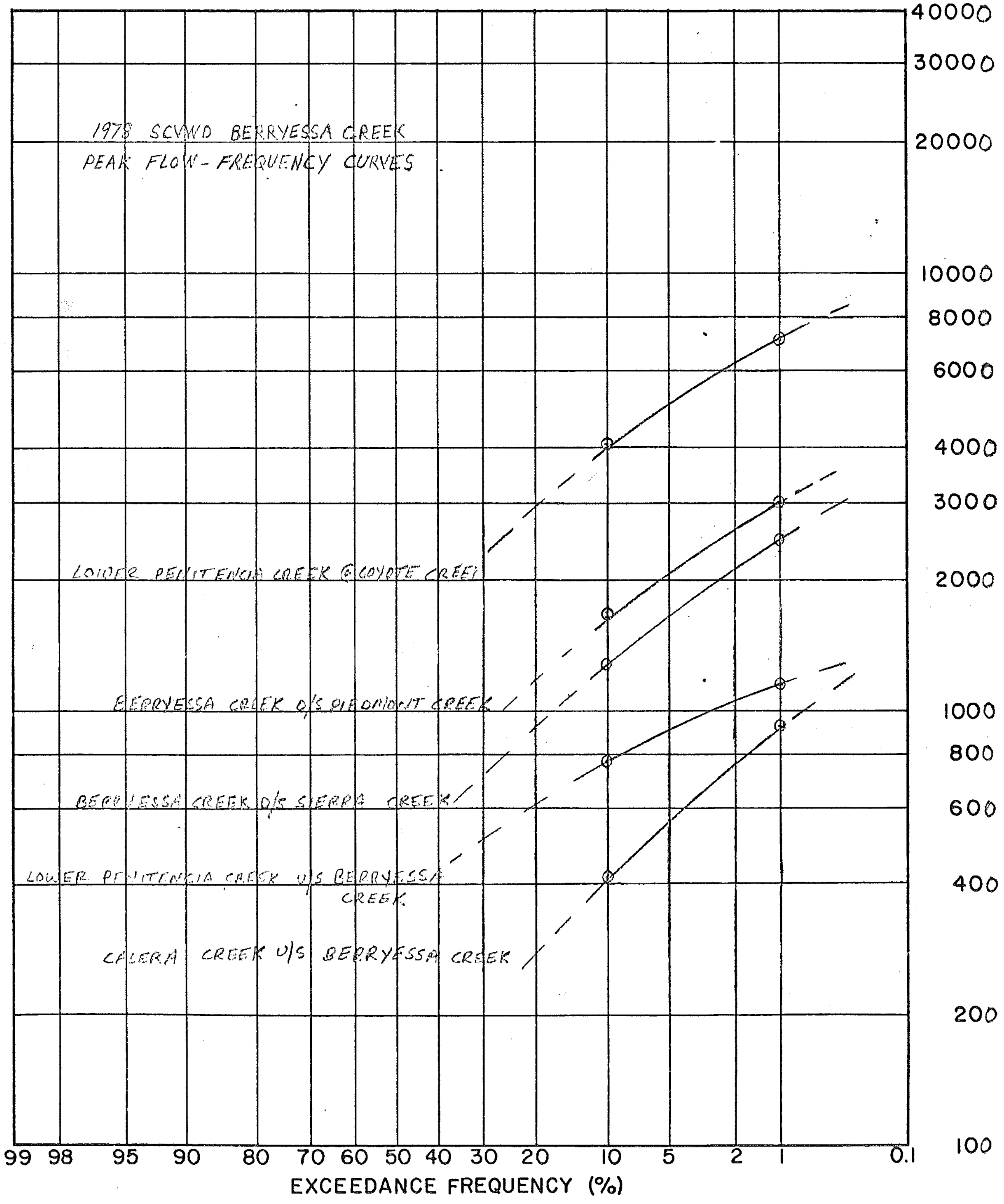

Figure 4-2

TABLE 4-2

BERRYESSA CREEK

Recommended 1% Design Flows

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	1580.	956.	539.	348.	4.51	
ROUTED TO	1	1580.	956.	539.	348.	4.51	Berryessa @ Piedmont Rd.
ROUTED TO	1	1591.	956.	539.	348.	4.51	
HYDROGRAPH AT	1	242.	109.	60.	38.	0.49	
ROUTED TO	1	242.	109.	60.	38.	0.49	
ROUTED TO	2	190.	109.	60.	38.	0.49	Sweigert u/s Berryessa Creek
2 COMBINED	2	1782.	1062.	599.	387.	5.00	Berryessa d/s Sweigert Creek
ROUTED TO	2	1775.	1062.	599.	387.	5.00	
HYDROGRAPH AT	3	451.	232.	130.	83.	0.93	
ROUTED TO	3	451.	232.	130.	83.	0.93	
ROUTED TO	3	254.	225.	130.	83.	0.93	Crosley u/s Berryessa Creek
2 COMBINED	3	2029.	1287.	728.	470.	5.93	Berryessa d/s Crosley Creek
ROUTED TO	4	1980.	1286.	728.	470.	5.93	
HYDROGRAPH AT	1	355.	182.	100.	64.	0.82	
ROUTED TO	1	355.	182.	100.	64.	0.82	
ROUTED TO	16	193.	175.	100.	64.	0.82	Sierra @ Piedmont Rd.
ROUTED TO	2	193.	174.	99.	64.	0.82	
HYDROGRAPH AT	2	118.	66.	36.	23.	0.33	
HYDROGRAPH AT	2	98.	72.	46.	31.	0.32	
2 COMBINED	2	210.	137.	82.	54.	0.65	
ROUTED TO	2	133.	131.	82.	54.	0.65	
2 COMBINED	17	326.	303.	182.	118.	1.47	Sierra @ Hostetter Rd.
ROUTED TO	3	326.	302.	182.	118.	1.47	
HYDROGRAPH AT	3	44.	24.	13.	9.	0.12	
HYDROGRAPH AT	3	56.	41.	26.	17.	0.18	
2 COMBINED	3	97.	65.	40.	26.	0.30	
ROUTED TO	3	86.	65.	40.	26.	0.30	
2 COMBINED	4	412.	366.	221.	144.	1.77	Sierra u/s Berryessa Creek
2 COMBINED	4	2389.	1641.	948.	614.	7.70	Berryessa d/s Sierra Creek
ROUTED TO	4	2392.	1641.	949.	614.	7.70	
HYDROGRAPH AT	4	40.	20.	11.	7.	0.10	
HYDROGRAPH AT	4	73.	44.	28.	16.	0.19	
2 COMBINED	4	112.	64.	39.	25.	0.29	
2 COMBINED	4	2471.	1704.	987.	639.	7.99	Berryessa @ Cropley Road
ROUTED TO	5	2376.	1702.	986.	639.	7.99	
HYDROGRAPH AT	5	81.	46.	25.	16.	0.23	
HYDROGRAPH AT	5	164.	121.	77.	51.	0.54	
2 COMBINED	5	238.	165.	102.	67.	0.77	
ROUTED TO	5	177.	164.	102.	67.	0.77	
2 COMBINED	5	2552.	1866.	1088.	707.	8.76	Berryessa @ Landess Ave.
ROUTED TO	6	2525.	1854.	1088.	707.	8.76	
HYDROGRAPH AT	6	16.	8.	4.	3.	0.04	
HYDROGRAPH AT	6	122.	75.	47.	30.	0.32	
2 COMBINED	6	137.	82.	51.	33.	0.36	
ROUTED TO	6	49.	48.	45.	33.	0.36	
2 COMBINED	6	2573.	1911.	1131.	740.	9.12	Berryessa @ Ames Ave.
ROUTED TO	7	2568.	1911.	1131.	740.	9.12	Berryessa u/s Piedmont Creek
HYDROGRAPH AT	1	345.	172.	95.	61.	0.88	
ROUTED TO	7	327.	172.	94.	61.	0.88	
ROUTED TO	7	300.	171.	94.	61.	0.88	
HYDROGRAPH AT	7	122.	69.	38.	24.	0.34	
HYDROGRAPH AT	3	220.	162.	103.	69.	0.71	
2 COMBINED	7	332.	229.	141.	94.	1.05	
ROUTED TO	7	314.	212.	141.	93.	1.05	
2 COMBINED	7	511.	374.	235.	154.	1.93	Piedmont u/s Berryessa Creek

TABLE 4-2 (cont_nued)

2 COMBINED	7	2982.	2285.	1365.	894.	11.05	Berryessa d/s Piedmont Creek
ROUTED TO	8	2967.	2284.	1365.	894.	11.05	
HYDROGRAPH AT	8	1420.	843.	470.	304.	4.02	
ROUTED TO	8	1420.	843.	470.	304.	4.02	Arroyo de los Coches u/s Berryessa Creek
2 COMBINED	8	4023.	3113.	1833.	1197.	15.07	Berryessa d/s los Coches Creek
ROUTED TO	9	4022.	3113.	1833.	1197.	15.07	
HYDROGRAPH AT	9	240.	128.	71.	46.	0.64	
ROUTED TO	9	240.	128.	71.	46.	0.64	
HYDROGRAPH AT	9	257.	133.	73.	47.	0.65	
HYDROGRAPH AT	9	103.	76.	48.	32.	0.33	
2 COMBINED	9	357.	206.	121.	79.	0.98	
ROUTED TO	9	166.	165.	121.	79.	0.98	
2 COMBINED	18	402.	288.	191.	125.	1.62	Tularcitos @ I-680
ROUTED TO	9	377.	288.	191.	125.	1.62	
HYDROGRAPH AT	9	99.	51.	28.	18.	0.25	
HYDROGRAPH AT	9	28.	21.	13.	9.	0.09	
2 COMBINED	9	126.	71.	41.	27.	0.34	
ROUTED TO	9	71.	68.	41.	27.	0.34	
2 COMBINED	9	448.	356.	232.	151.	1.96	Tularcitos u/s Berryessa Creek
2 COMBINED	9	4454.	3469.	2065.	1349.	17.03	Berryessa d/s Tularcitos Creek
ROUTED TO	10	4430.	3466.	2064.	1349.	17.03	
HYDROGRAPH AT	10	162.	83.	45.	29.	0.42	
HYDROGRAPH AT	10	39.	29.	18.	12.	0.13	
2 COMBINED	10	200.	111.	64.	41.	0.55	
ROUTED TO	10	53.	53.	52.	41.	0.55	
2 COMBINED	10	4483.	3518.	2112.	1390.	17.58	Berryessa d/s Drainage Lagoon
ROUTED TO	11	4472.	3518.	2112.	1390.	17.58	
HYDROGRAPH AT	11	120.	84.	45.	29.	0.43	
HYDROGRAPH AT	11	336.	244.	154.	103.	1.11	
2 COMBINED	11	429.	328.	200.	132.	1.54	
ROUTED TO	11	424.	328.	200.	132.	1.54	Ford & Wrigley Ditches u/s Berryessa Creek
2 COMBINED	11	4884.	3844.	2311.	1522.	19.12	Berryessa d/s Ford & Wrigley Ditches
ROUTED TO	12	4883.	3844.	2311.	1522.	19.12	
HYDROGRAPH AT	19	893.	520.	284.	183.	2.43	Calera @ I-680
ROUTED TO	1	893.	520.	284.	183.	2.43	
ROUTED TO	2	854.	518.	284.	183.	2.43	
HYDROGRAPH AT	2	91.	49.	26.	17.	0.24	
HYDROGRAPH AT	2	82.	59.	37.	25.	0.26	
2 COMBINED	2	168.	107.	64.	42.	0.50	
ROUTED TO	2	71.	70.	61.	42.	0.50	
2 COMBINED	12	922.	586.	340.	225.	2.93	Calera u/s Berryessa Creek

TABLE 4-2 (continued)

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                     RUNOFF SUMMARY, AVERAGE FLOW

                           PEAK    6-HOUR   24-HOUR   72-HOUR      AREA
HYDROGRAPH AT      12     5642.     4408.     2651.     1744.     22.05  Berryessa d/s Calera Creek
HYDROGRAPH AT       1      229.      165.       90.       58.      0.85
HYDROGRAPH AT       1      257.      187.      118.       79.      0.85
 2 COMBINED         1      448.      353.      208.      137.      1.70
ROUTED TO           1      341.      336.      208.      137.      1.70
ROUTED TO          20      341.      335.      208.      137.      1.70  East Penitencia @ Trimble Road
HYDROGRAPH AT       1      287.      217.      118.       77.      1.17
HYDROGRAPH AT       1      227.      165.      104.       69.      0.78
 2 COMBINED         1      468.      382.      223.      146.      1.95
ROUTED TO           1      467.      381.      223.      146.      1.95
ROUTED TO          20      466.      381.      222.      146.      1.95  Lower Penitencia @ Trimble Road
 2 COMBINED        20      805.      709.      430.      283.      3.65  Lower Penitencia d/s East Penitencia Creek
ROUTED TO           3      805.      709.      430.      283.      3.65
HYDROGRAPH AT       3       36.       27.       14.        9.      0.14
HYDROGRAPH AT       3       58.       43.       27.       18.      0.20
 2 COMBINED         3       87.       69.       41.       27.      0.34
ROUTED TO           3       87.       69.       41.       27.      0.34
 2 COMBINED        21      887.      776.      471.      310.      3.99  Lower Penitencia @ Curtis Ave.
ROUTED TO           4      872.      771.      470.      310.      3.99
HYDROGRAPH AT       4      137.      102.       56.       36.      0.54
HYDROGRAPH AT       4      186.      135.       86.       57.      0.63
 2 COMBINED         4      295.      238.      141.       93.      1.17
ROUTED TO           4      280.      237.      141.       93.      1.17
 2 COMBINED        13     1152.     1000.      611.      403.      5.16  Lower Penitencia u/s Berryessa Creek
ROUTED TO          13     1153.     1000.      611.      403.      5.16
 2 COMBINED        13     6737.     5390.     3260.     2148.     27.21  Lower Penitencia d/s Berryessa Creek
ROUTED TO          14     6722.     5398.     3260.     2148.     27.21
HYDROGRAPH AT      14      116.       64.       35.       23.      0.32
HYDROGRAPH AT      14      151.      111.       71.       47.      0.49
 2 COMBINED        14      262.      173.      106.       70.      0.81
ROUTED TO          14      240.      173.      106.       70.      0.81  Sunny Hills Outfall u/s L. Penitencia Creek
 2 COMBINED        14     6962.     5558.     3365.     2217.     28.02  L. Penitencia d/s Sunny Hills Outfall
HYDROGRAPH AT      14       62.       27.       13.        8.      0.24
ROUTED TO          14       37.       29.       13.        8.      0.24
 2 COMBINED        14     6997.     5583.     3377.     2225.     28.26
ROUTED TO          15     6977.     5582.     3377.     2225.     28.26  Lower Penitencia u/s Coyote Creek
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TABLE 4-3 (Continued)

BERRYESSA CREEK

Recommended 10% Design Flows

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	700.	399.	239.	155.	4.51	
ROUTED TO	1	700.	399.	239.	155.	4.51	Berryessa @ Piedmont Rd.
ROUTED TO	1	672.	399.	239.	155.	4.51	
HYDROGRAPH AT	1	111.	57.	27.	17.	0.49	
ROUTED TO	1	111.	57.	27.	17.	0.49	
ROUTED TO	2	109.	57.	27.	17.	0.49	Sweigert u/s Berryessa Creek
2 COMBINED	2	772.	454.	266.	172.	5.00	Berryessa d/s Sweigert Creek
ROUTED TO	2	772.	454.	266.	172.	5.00	
HYDROGRAPH AT	3	191.	113.	55.	35.	0.93	
ROUTED TO	3	191.	113.	55.	35.	0.93	
ROUTED TO	3	187.	113.	55.	35.	0.93	Crosley u/s Berryessa Creek
2 COMBINED	3	947.	564.	321.	208.	5.93	Berryessa d/s Crosley Creek
ROUTED TO	4	925.	564.	321.	208.	5.93	
HYDROGRAPH AT	1	151.	88.	42.	27.	0.82	
ROUTED TO	1	151.	88.	42.	27.	0.82	
ROUTED TO	/6	143.	87.	42.	27.	0.82	Sierra @ Piedmont Rd.
ROUTED TO	2	137.	87.	42.	27.	0.82	
HYDROGRAPH AT	2	77.	39.	18.	11.	0.33	
HYDROGRAPH AT	2	66.	48.	30.	20.	0.32	
2 COMBINED	2	139.	86.	48.	31.	0.65	
ROUTED TO	2	130.	85.	47.	31.	0.65	
2 COMBINED	/7	266.	172.	89.	58.	1.47	Sierra @ Hostetter Rd.
ROUTED TO	3	262.	171.	89.	58.	1.47	
HYDROGRAPH AT	3	28.	14.	7.	4.	0.12	
HYDROGRAPH AT	3	38.	27.	17.	11.	0.18	
2 COMBINED	3	64.	41.	24.	15.	0.30	
ROUTED TO	3	62.	41.	24.	15.	0.30	
2 COMBINED	4	322.	213.	113.	73.	1.77	Sierra u/s Berryessa Creek
2 COMBINED	4	1234.	775.	433.	281.	7.70	Berryessa d/s Sierra Creek
ROUTED TO	4	1232.	775.	433.	281.	7.70	
HYDROGRAPH AT	4	26.	12.	5.	3.	0.10	
HYDROGRAPH AT	4	50.	29.	18.	12.	0.19	
2 COMBINED	4	75.	41.	23.	15.	0.29	
2 COMBINED	4	1285.	816.	456.	296.	7.99	Berryessa @ Cropley Rd.
ROUTED TO	5	1227.	814.	456.	296.	7.99	
HYDROGRAPH AT	5	52.	27.	12.	8.	0.23	
HYDROGRAPH AT	5	111.	80.	50.	33.	0.54	
2 COMBINED	5	158.	106.	62.	41.	0.77	
ROUTED TO	5	152.	106.	62.	41.	0.77	
2 COMBINED	5	1374.	919.	518.	337.	8.76	Berryessa @ Landess Ave.
ROUTED TO	6	1361.	919.	518.	337.	8.76	
HYDROGRAPH AT	6	11.	5.	2.	1.	0.04	
HYDROGRAPH AT	6	84.	50.	30.	20.	0.32	
2 COMBINED	6	93.	54.	32.	21.	0.36	
ROUTED TO	6	44.	43.	32.	21.	0.36	
2 COMBINED	6	1405.	962.	550.	358.	9.12	Berryessa @ Ames Ave.
ROUTED TO	7	1399.	961.	549.	358.	9.12	Berryessa u/s Piedmont Creek
HYDROGRAPH AT	1	161.	91.	43.	28.	0.88	
ROUTED TO	7	160.	91.	43.	28.	0.88	
ROUTED TO	7	155.	91.	43.	28.	0.88	
HYDROGRAPH AT	7	79.	40.	19.	12.	0.34	
HYDROGRAPH AT	3	147.	137.	67.	44.	0.71	
2 COMBINED	7	219.	146.	88.	[illegible]	1.05	
ROUTED TO	7	210.	146.	85.	56.	1.05	
2 COMBINED	7	363.	235.	128.	84.	1.93	Piedmont u/s Berryessa Creek

TABLE 4-3 (continued)

2 COMBINED	7	1719.	1197.	678.	441.	11.05	Berryessa d/s Piedmont Creek
ROUTED TO	8	1712.	1196.	678.	441.	11.05	
HYDROGRAPH AT	8	615.	403.	202.	130.	4.02	
ROUTED TO	8	615.	403.	202.	130.	4.02	Arroyo de los Coches u/s Berryessa Creek
2 COMBINED	8	2273.	1596.	879.	571.	15.07	Berryessa d/s los Coches Creek
ROUTED TO	9	2255.	1595.	879.	571.	15.07	
HYDROGRAPH AT	9	120.	64.	30.	19.	0.64	
ROUTED TO	9	170.	64.	30.	19.	0.64	
HYDROGRAPH AT	9	167.	78.	36.	23.	0.65	
HYDROGRAPH AT	9	70.	50.	31.	21.	0.33	
2 COMBINED	9	235.	127.	67.	44.	0.98	
ROUTED TO	9	161.	126.	67.	44.	0.98	
2 COMBINED	18	280.	189.	97.	63.	1.62	Tularcitos @ I-680
ROUTED TO	9	269.	189.	97.	63.	1.62	
HYDROGRAPH AT	9	64.	30.	14.	9.	0.25	
HYDROGRAPH AT	9	19.	14.	8.	6.	0.09	
2 COMBINED	9	83.	43.	22.	14.	0.34	
ROUTED TO	9	70.	43.	22.	14.	0.34	
2 COMBINED	9	340.	231.	119.	77.	1.96	Tularcitos u/s Berryessa Creek
2 COMBINED	9	2578.	1826.	998.	649.	17.03	Berryessa d/s Tularcitos Creek
ROUTED TO	10	2552.	1825.	998.	649.	17.03	
HYDROGRAPH AT	10	106.	49.	22.	14.	0.42	
HYDROGRAPH AT	10	27.	19.	12.	8.	0.13	
2 COMBINED	10	132.	67.	34.	22.	0.55	
ROUTED TO	10	51.	51.	34.	22.	0.55	
2 COMBINED	10	2603.	1874.	1032.	671.	17.58	Berryessa d/s Drainage Lagoon
ROUTED TO	11	2589.	1873.	1031.	671.	17.58	
HYDROGRAPH AT	11	72.	48.	22.	14.	0.43	
HYDROGRAPH AT	11	223.	161.	100.	66.	1.11	
2 COMBINED	11	277.	209.	122.	81.	1.54	
ROUTED TO	11	274.	208.	122.	81.	1.54	Ford & Wrigley Ditches u/s Berryessa Creek
2 COMBINED	11	2850.	2081.	1153.	752.	19.12	Berryessa d/s Ford & Wrigley Ditches
ROUTED TO	12	2840.	2091.	1153.	752.	19.12	
HYDROGRAPH AT	19	410.	217.	130.	84.	2.43	Calera @ I-680
ROUTED TO	1	410.	217.	130.	84.	2.43	
ROUTED TO	2	363.	216.	130.	84.	2.43	
HYDROGRAPH AT	2	57.	28.	13.	8.	0.24	
HYDROGRAPH AT	2	54.	39.	24.	16.	0.26	
2 COMBINED	2	108.	66.	37.	24.	0.50	
ROUTED TO	2	66.	64.	37.	24.	0.50	
2 COMBINED	12	428.	275.	167.	109.	2.93	Calera u/s Berryessa Creek
2 COMBINED	12	3154.	2354.	1318.	860.	22.05	Berryessa d/s Calera Creek

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TABLE 4-3 (continued)

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                        RUNOFF SUMMARY, AVERAGE FLOW

                             PEAK    6-HOUR   24-HOUR   72-HOUR      AREA
HYDROGRAPH AT        1      3154.     2354.     1318.      860.     22.05   Berryessa d/s Calera Creek
HYDROGRAPH AT        1       138.       95.       44.       28.      0.85
HYDROGRAPH AT        1       171.      123.       76.       51.      0.85
 2 COMBINED          1       283.      218.      120.       79.      1.70
ROUTED TO            1       278.      217.      120.       79.      1.70
ROUTED TO           20       277.      216.      120.       79.      1.70   East Penitencia @ Trimble Rd.
HYDROGRAPH AT        1       172.      124.       59.       38.      1.17
HYDROGRAPH AT        1       150.      108.       67.       45.      0.78
 2 COMBINED          1       291.      232.      126.       82.      1.95
ROUTED TO            1       290.      232.      126.       82.      1.95
ROUTED TO           20       290.      232.      126.       82.      1.95   Lower Penitencia @ Trimble Rd.
 2 COMBINED         20       567.      449.      246.      162.      3.65   Lower Penitencia d/s East Penitencia Creek
ROUTED TO            3       566.      448.      246.      162.      3.65
HYDROGRAPH AT        3        22.       15.        7.        5.      0.14
HYDROGRAPH AT        3        39.       28.       17.       11.      0.20
 2 COMBINED          3        55.       43.       24.       16.      0.34
ROUTED TO            3        55.       43.       24.       16.      0.34
 2 COMBINED         21       618.      491.      270.      178.      3.99   Lower Penitencia @ Curtis Ave.
ROUTED TO            4       600.      486.      270.      178.      3.99
HYDROGRAPH AT        4        82.       58.       27.       18.      0.54
HYDROGRAPH AT        4       123.       89.       55.       37.      0.63
 2 COMBINED          4       186.      147.       82.       54.      1.17
ROUTED TO            4       185.      147.       82.       54.      1.17
 2 COMBINED         13       773.      630.      352.      232.      5.16
ROUTED TO           13       773.      630.      352.      232.      5.16   Lower Penitencia u/s Berryessa Creek
 2 COMBINED         13      3896.     2972.     1669.     1092.     27.21   Lower Penitencia d/s Berryessa Creek
ROUTED TO           14      3894.     2972.     1669.     1092.     27.21
HYDROGRAPH AT       14        76.       38.       17.       11.      0.32
HYDROGRAPH AT       14       102.       73.       46.       30.      0.49
 2 COMBINED         14       175.      110.       63.       41.      0.81
ROUTED TO           14       169.      110.       63.       41.      0.81   Sunny Hills Outfall u/s L. Penitencia Creek
 2 COMBINED         14      4019.     3081.     1732.     1134.     28.02   Lower Penitencia d/s Sunny Hills Outfall
HYDROGRAPH AT       14        65.       27.       13.        8.      0.24
ROUTED TO           14        38.       26.       13.        8.      0.24
 2 COMBINED         14      4052.     3106.     1744.     1142.     28.26
ROUTED TO           15      4043.     3104.     1744.     1142.     28.26   Lower Penitencia u/s Coyote Creek
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